AD-A144 190 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS GLEN LAKE DAM (CT 003..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV AUG 79 1/2

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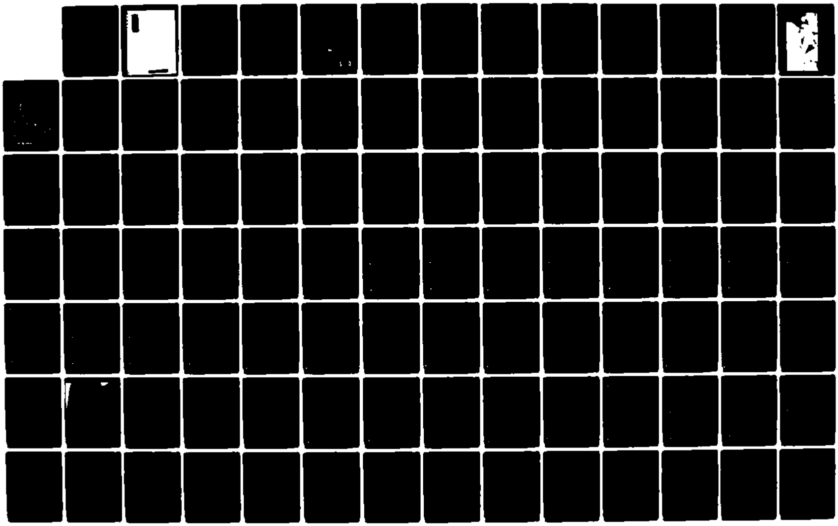

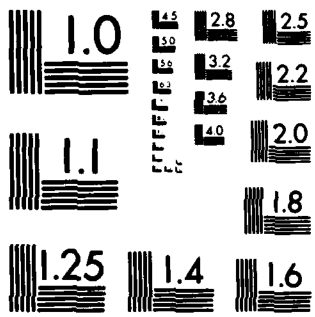


MICROCOPY RESOLUTION TEST CHART
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| REPORT DOCUMENTATION PAGE | | READ INSTRUCTIONS BEFORE COMPLETING FORM |
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18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut Coastal Basin
Woodbridge, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The 380± foot long dam is a concrete gravity section rising approximately 62 feet above the bed of the Sargent River. Based on the visual inspection at the site and past performance, the dam appears to be in good condition. Based on the size (intermediate) and the hazard classification (high) of the dam determined in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to the Probable Maximum Flood.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED

NOV 28 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Glen Lake Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, New Haven Water Company.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely,

Max B. Scheider

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

CONNECTICUT COASTAL BASIN

WOODBRIDGE, CONNECTICUT

# GLEN LAKE DAM
# CT 00317

## PHASE I INSPECTION REPORT
## NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154



AUGUST, 1979

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

| | |
|---|---|
| Name of Dam: | GLEN LAKE DAM |
| Inventory Number: | CT - 00317 |
| State Located: | CONNECTICUT |
| County Located: | NEW HAVEN |
| Town Located: | WOODBRIDGE |
| Stream: | SARGENT RIVER |
| Owner: | NEW HAVEN WATER COMPANY |
| Date of Inspection: | MAY 1, 1979 |
| Inspection Team: | PETER M. HEYNEN, P.E. |
| | CALVIN GOLDSMITH |
| | MIRON PETROVSKY |
| | GEORGE STEPHENS |
| | AL BUCHER |

The 380$\pm$ foot long dam is a concrete gravity section rising approximately 62 feet above the bed of the Sargent River. The dam is founded on bedrock with the deepest foundation extending to 75 feet below the top of the dam. The spillway at the right end of the dam, is a concrete ogee section 40 feet in width, with 9 feet of freeboard from the crest of the spillway to the top of the dam.

The outlets, regulated by a gatehouse on the upstream face of the dam, consist of a 24 inch supply main to the filtration plant downstream of Lake Dawson, a 30 inch low level outlet, and a 10 inch intake well drain, both of which discharge to the natural streambed near the center of the dam.

Based on the visual inspection at the site and past performance, the dam appears to be in good condition. No evidence of instability was observed in the dam or its appurtenances.

Based on the size (Intermediate) and the hazard classification (High) of the dam determined in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to the Probable Maximum Flood (PMF). Peak outflow is 8,220 cfs with the dam overtopped 1.6 feet. Based on our hydraulic computations, the spillway capacity is 4,100 cfs, which is equivalent to approximately 50% of the routed test flood outflow.

It is recommended that further studies by a qualified professional engineer be initiated by the owner to perform a more refined hydraulic/hydrologic study to determine the spillway capacity and overtopping potential. Recommendations should be made by the engineer and implemented by the owner to increase the project discharge based upon the refined hydraulic/-hydrologic study.

The above recommendations, and any required remedial measures, are discussed in Section 7 and should be instituted within 2 years of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

This Phase I Inspection Report on Glen Lake Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Joseph W. Finegan

JOSEPH W. FINEGAN, JR., MEMBER
Water Control Branch
Engineering Division

Joseph A. McElroy

JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, CHAIRMAN
Chief, Structural Section
Design Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

## PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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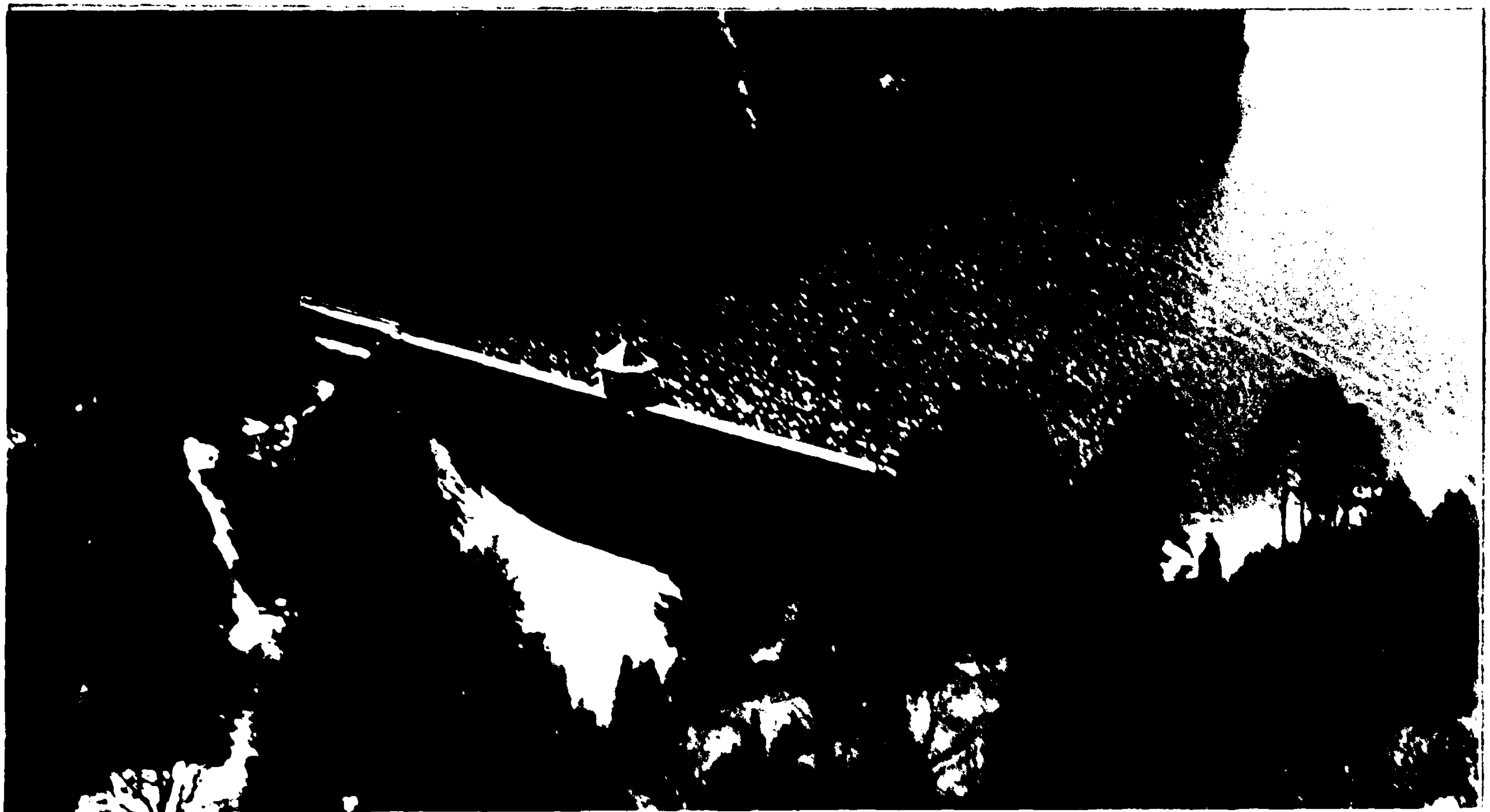
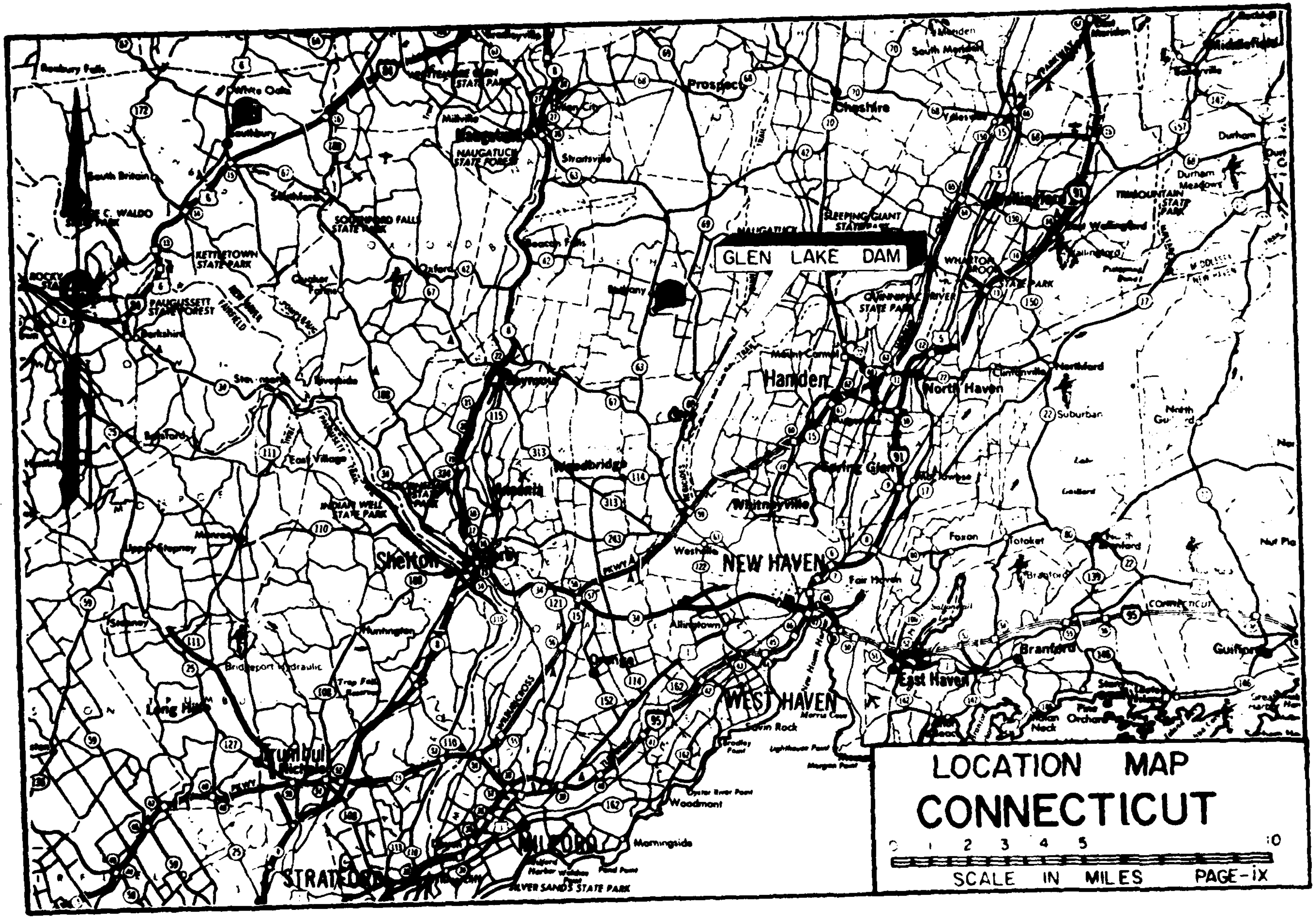
## APPENDICES



OVERVIEW PHOTO

U.S. ARMY ENGINEER ... NEW ENGLAND ...


GLEN LAKE DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5 10
SCALE IN MILES
PAGE-IX
NEW HAVEN
WEST HAVEN
Hamden
North Haven
East Haven
Branford
Guilford
Cheshire
Prospect
Shelton
Trumbull
Stratford
Milford
Woodbridge
Whitneyville
Westville
Fair Haven
Allingtown
Oxford
Southbury
South Britain
Naugatuck
NAUGATUCK STATE FOREST
SLEEPING GIANT STATE PARK
KETTLETOWN STATE PARK
PAUGUSSETT STATE FOREST
INDIAN WELL STATE PARK
SILVER SANDS STATE PARK
TRIMOUNTAIN STATE PARK
Durham
Durham Meadows
South Meriden
Meriden
Yalesville
Wallingford
East Wallingford
Northford
Suburban
North Guilford
Foxon
Totoket
North Branford
Bethany
Bridgeport Hydraulic
Long Hill
Huntington
Stratsville
Beacon Falls
Millville
White Oaks
East Village
Stevenson
Riverside
Botsford
Upper Stepney
Monroe
Stepney
Mount Carmel
Centerville
Spring Glen
Morris Cove
Savin Rock
Woodmont
Morningside
Oyster River Point
Short Beach
Pine Orchard
Indian Neck

PHASE I INSPECTION REPORT

GLEN LAKE DAM

SECTION I - PROJECT INFORMATION

## 1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of March 30, 1979 from John P. Chandler, Colonel, Corps of Engineers. Contract No. DACW 33-79-3-0059 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

## 1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the Sargent River in a rural section of the town of Woodbridge, County of New Haven, State of Connecticut. The dam is shown on the Mount Carmel U.S.G.S. Quadrangle Map as having coordinates latitude N 41$^{\circ}$ 22.6' and longitude W 72$^{\circ}$ 58.7'.

b. Description of Dam and Appurtenances - The dam is 380$\pm$ feet long and the width at the crest is 9 feet, including the coping overhang on the downstream side. The top of the dam is about 62 feet above the bed of the Sargent River. The dam is a rubble concrete gravity section with "large stones" placed in the concrete, and is founded on bedrock. The upstream face is near vertical, while the downstream face curves from vertical at the top to an inclination of 6 horizontal to 10 vertical at the toe. Gunite resurfacing of the dam was performed in 1948, covering the 3 inch overhang on the top of the downstream face.

The spillway is a concrete ogee section located at the right end of the dam. It is 40 feet in width and founded on bedrock. Flow from the spillway discharges over a vertical drop to a bedrock, sand and gravel channel. The freeboard from the spillway crest to the top of the dam is 9 feet.

The outlets, located to the right of the center of the dam, consist of a 30 inch cast iron low level outlet, and a 24 inch cast iron supply main which feeds the filtration plant below Lake Dawson on the West River. There are three (3) intake windows which direct flow into a wet well and to the supply main, and are protected by removable screens. In addition, there is a 10 inch outlet to drain the intake well. The gate valves for all intakes and outlets are manually operated from a gate house on the upstream side of the dam. All gate valves are operable.

c. Size Classification - INTERMEDIATE - The dam impounds an estimated 710 acre-feet of water with the lake level at the top of the dam, which at elevation 227.3 is about 62 feet above the old streambed. According to the Recommended Guidelines, a dam with a height of between 40 and 100 feet is classified as intermediate in size.

d. <u>Hazard Classification</u> - HIGH - Glen Lake Dam is located upstream from Lake Dawson and Konolds Pond, and low lying urban developments of the Westville section of New Haven. There is one low lying house immediately downstream of Glen Lake Dam, and one low lying house and a filtration plant immediately downstream of Lake Dawson Dam. There are also at least 10 low lying residential and business structures along the shore of Konolds Pond, and many more downstream of Konold's Pond Dam in Westville.

e. <u>Ownership</u> - New Haven Water Company
90 Sargent Drive
New Haven, CT 06511
Mr. Jack Reynolds (203) 624-6671

f. <u>Operator</u> - Mr. Ken Seaton
New Haven Water Company
(203) 393-1619

g. <u>Purpose of Dam</u> - Public Water Supply

h. <u>Design and Construction History</u> - The following information is believed to be accurate based on the plans and correspondence available. The dam was constructed in 1906-1907 by the New Haven Water Company. Albert B. Hill was the engineer and the New York Continental Jewell Filtration Company and Upson & Grannis were the contractors. In 1948, gunite repairs of the dam and spillway surfaces were performed with Clarence M. Blair Associates as the engineer and the Cement Gun Company as the contractor. In 1968-1969 the spillway was lowered 5 feet and the entrance channel to the spillway widened and deepened. Malcolm Pirnie Engineers were the engineers and C. W. Blakeslee and Sons, Inc., were the contractors. The roof of the gate house was replaced in 1976.

i. <u>Normal Operational Procedures</u> - The 24 inch supply main outlet is opened as needed for water supply purposes. The various level inlet gates are opened as necessary to maintain water quality, based on the results of tests on water samples from different depths of the lake. The low level outlet is opened once every year for several hours to flush it out.

## 1.3 <u>PERTINENT DATA</u>

a. <u>Drainage Area</u> - 5.7 square miles of rolling, wooded terrain of which 1.7 square miles is direct to Glen Lake and 4.0 square miles drains into Lake Chamberlain on the Sargent River, outflow from which feeds Glen Lake.

b. Discharge at Damsite - Discharge from the lake is through a 24 inch supply main, a 30 inch low level outlet, and a 10 inch intake well drain.

| | |
|---|---|
| 1. Outlet Works (Conduits): | 24 inch water supply pipe at el. 167.5 pipe invert (lowest intake window to wet well at el. 176.3)<br><br>30 inch low level outlet at invert el. 167.3 (Approx.)<br><br>10 inch well drain at invert el. 166 (Approx.) |
| 2. Maximum known flood at damsite: | 2'7" above spillway crest (Oct., 1955 - prior to spillway lowering) |
| 3. Ungated spillway capacity @ top of dam el. 227.3: | 4100 cfs. |
| 4. Ungated spillway capacity @ test flood el.: | N/A |
| 5. Gated spillway capacity @ normal pool el.: | N/A |
| 6. Gated spillway capacity @ test flood el.: | N/A |
| 7. Total spillway capacity @ test flood el.: | N/A |
| 8. Total project discharge @ test flood el. 228.9: | 8220 cfs. |

c. Elevations (Feet Above Mean Sea Level)

| | |
|---|---|
| 1. Streambed at centerline of dam: | $165.3^{\pm}$ |
| 2. Maximum tailwater: | N/A |
| 3. Upstream portal invert diversion tunnel: | N/A |
| 4. Recreation pool: | N/A |

| | | |
|---|---|---|
| 5. | Full flood control pool: | N/A |
| 6. | Spillway crest: | 218.3 |
| 7. | Design surcharge (original design): | N/A |
| 8. | Top of Dam: | 227.3 |
| 9. | Test flood design surcharge: | 228.9 |

d. <u>Reservoir</u>

| | | |
|---|---|---|
| 1. | Length of maximum pool: | 2500± ft. |
| 2. | Length of recreation pool: | N/A |
| 3. | Length of flood control pool: | N/A |

e. <u>Storage</u>

| | | |
|---|---|---|
| 1. | Recreation pool: | N/A |
| 2. | Flood control pool: | N/A |
| 3. | Spillway crest pool: | 482 acre-ft. |
| 4. | Top of dam: | 710 acre-ft. |
| 5. | Test flood pool: | 755± acre-ft. |

f. <u>Reservoir Surface</u>

| | | |
|---|---|---|
| 1. | Recreation pool: | N/A |
| 2. | Flood control pool: | N/A |
| 3. | Spillway crest: | 23.0 acres |
| 4. | Test flood pool: | N/A |
| 5. | Top of dam: | 28± acres |

g. <u>Dam</u>

| | | |
|---|---|---|
| 1. | Type: | Concrete gravity section |

| | |
|---|---|
| 2. Length: | 380± ft. |
| 3. Height: | 62 ft. to streambed<br>75 ft. to foundation |
| 4. Top width: | 9 ft. |
| 5. Side slopes: | Vertical (Upstream)<br>6H to 10V (Downstream) |
| 6. Zoning: | N/A |
| 7. Impervious Core: | N/A |
| 8. Cutoff: | Bedrock |
| 9. Grout curtain: | N/A |
| 10. Other: | N/A |

h. <u>Diversion and Regulating Tunnel</u> - N/A

i. <u>Spillway</u>

| | |
|---|---|
| 1. Type: | Concrete ogee section |
| 2. Length of weir: | 40 ft. |
| 3. Crest elevation: | 218.3 |
| 4. Gates: | None |
| 5. Upstream Channel: | Gently sloping |
| 6. Downstream Channel: | Gently sloping concrete apron to vertical rock ledge drop-off. |
| 7. General: | Gravel streambed |

j. <u>Regulating Outlets</u>
<u>24 inch outlet</u>

| | |
|---|---|
| 1. Invert: | Lowest intake to intake well el. 176.3 |
| 2. Size: | 24 inch diameter |
| 3. Description: | Cast iron pipe-supply main |
| 4. Control Mechanism: | Hand operated |

| | |
|---|---|
| 5. Other: | N/A |

<u>30 Inch outlet</u>

| | |
|---|---|
| 1. Invert: | 167.3 |
| 2. Size: | 30 inch |
| 3. Description: | Cast iron low level outlet pipe |
| 4. Control Mechanism: | Hand operated |
| 5. Other: | N/A |

<u>10 inch outlet</u>

| | |
|---|---|
| 1. Invert: | 166± |
| 2. Size: | 10 inch |
| 3. Description: | Cast iron intake well drain pipe |
| 4. Control Mechanism: | Hand operated |
| 5. Other: | N/A |

## SECTION 2: ENGINEERING DATA

### 2.1 DESIGN

a. Available Data - The available data consists of drawings, correspondence, records, and calculations by the State of Connecticut D.E.P., the New Haven Water Company, Joseph W. Cone, Malcolm Pirnie Engineers, Clarence Blair Associates, and Albert B. Hill. Pertinent data is included in Appendix B.

b. Design Features - The drawings and reports indicate the design features stated previously herein.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the original construction. Limited design information by Malcolm Pirnie Engineers for the 1968 lowering of the spillway is included in Appendix, Section B, including a cross-section of the dam, and a rough stability analysis diagram.

### 2.2 CONSTRUCTION

a. Available Data - The available construction data consists of drawings of the original dam by Albert B. Hill, and drawings of the altered gatehouse by Blair and Marchant, Inc. Drawings of elevations and cross-sections in the vicinity of Glen Dam by Clarence Blair Associates also obtained, reflect as-built conditions of parts of the dam.

b. Construction Considerations - Construction data was scarce, therefore no information pertaining to construction considerations was available.

### 2.3 OPERATIONS

Lake levels are taken daily. To our knowledge, the spillway capacity has never been exceeded. No other formal operations records were obtained.

### 2.4 EVALUATION

a. Availability - Existing data was provided by the owners and by the Connecticut D.E.P. The owner made the facility available for visual inspection.

b. Adequacy - The limited amount of detailed engineering data available was generally inadequate to perform an in-depth assessment of the dam, therefore, the final assessment of this dam must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity and approximate hydrologic judgement.

c. Validity - A comparison of records data and visual observations reveals no observable significant discrepancies in the record data.

## SECTION 3: VISUAL INSPECTION

a. General - The general condition of the dam is good. Inspection did reveal some areas requiring attention. The reservoir level was at elevation 218.5, approximately 0.2 feet above the crest of the spillway at the time of our inspection.

b. Dam:

Crest and Upstream Face - No misalignment, deterioration, spalling or cracking were observed on the crest, including zones along the expansion joints 50 to 75 feet apart. The vertical upstream face near the crest does not exhibit significant cracks or deterioration from freeze-thaw or other influences. (Photo 5).

Downstream Face/Slope - The curvilinear downstream face, with an inclination of 6 horizontal to 10 vertical near the bottom, was resurfaced with gunite in 1948. At the present time the gunite coating is deteriorated in many sections, especially in the areas of the old cracks, construction and expansion joints and seepage (Photos 7 & 8). There are drilled holes in the face approximately 6 inches deep and 1.5 inches in diameter which appear to be intended to relieve the water pressure on the downstream dam surface. Some of the holes had white lime deposits. No signs of downstream seepage through the foundation were observed. At the left side of the dam on the downstream slope adjacent to the toe of the exposed portion of the dam, there are at least two trees about 18 to 24 inches in diameter growing very close to the face of the dam (Photo 6). There was no seepage observed in the vicinity of these trees, however they do extend above the top of the dam up to 20 or 30 feet and could be overturned by high winds.

The right rock abutment adjacent to Dillon Road appears to be in good condition. The left abutment is a wooded natural ground area with downstream slopes at an inclination of approximately 1.5 horizontal to 1 vertical. The upstream left side of the reservoir bank adjacent to the left dam abutment is an earth dike that has a very broad crest and a maximum height of 6 feet (Photo 1). The crest is grass covered and has a small stand of conifers near the dam. The downstream face of the dike is a rough stone wall as depicted on Sheet B-1 in Appendix B (Photo 2). Below this wall, at a distance of 200 to 300 feet from the reservoir water line, a 20 to 30 foot wide wet area was discovered. This area has formed a brook with a flow rate of approximately 10 to 15 gallons per minute. The brook water was used for many years for the water supply of the nearest house downstream of the dam.

Spillway - The concrete ogee spillway spanned by a concrete service bridge is in good condition. The spillway and the upstream right wingwall, the side training walls and the short apron were repaired in 1969. The new reinforced concrete 3 foot high spillway weir which was anchored into the original concrete weir, is in good condition (Photo 4). The old concrete of the training walls and the apron has cracks with efflorescence, and 2 to 4 inch deep pockets which appear to have been worn or cavitated. Debris, including trees and stones, were noted on the weir and the downstream apron of the spillway.

c. Appurtenant Structures - The gate house, which has a new roof, has no signs of visible deterioration (Photo 5). The downstream outlet masonry headwall appears to be stable (Photo 3). The submerged wet well drain and low level outlet at the base of the wall could not be observed. The gate valve operating mechanisms appear well maintained.

d. Reservoir Area - The reservoir area is bordered on the right by Dillon Road. The area surrounding the reservoir is wooded and undeveloped.

e. Downstream Channel - The downstream channel is the natural bed of the Sargent River. It has a gravel and boulder bottom and a steep, stable rock and wooded right bank along the roadway. No substantial obstructions to the flow were detected.

3.2 EVALUATION

Based upon the visual inspection, it was possible to assess the dam as being generally in good condition. The following features which could influence the future condition and/or stability of the dam were identified:

1. The extensive wet area on the slope downstream of the left dike should be monitored periodically.

2. The deteriorated concrete areas of the spillway, the spillway training walls and the downstream face of the gravity section will be subject to further deterioration of the concrete if not repaired.

3. The trees on the downstream earth slope adjacent to the left side of the downstream face of the dam could be subject to overturning by the wind, which has potential for causing damage which would affect the stability of the dam.

## SECTION 4: OPERATIONAL PROCEDURES

### 4.1 REGULATING PROCEDURES

No formal regulating procedures exist for this dam other than those necessary for providing sufficient water for public water supply purposes.

### 4.2 MAINTENANCE OF DAM

Water levels in the lake are recorded daily and water samples for chemical analysis are taken bi-weekly. Grass downstream from the dam is cut regularly during the growing season. Debris is removed from the spillway as needed, piled beside the spillway apron, and removed once a year.

A yearly inspection program was instituted by the New Haven Water Company three years ago encompassing all their dams and is performed by a consultant qualified in the field of dam inspection.

### 4.3 MAINTENANCE OF OPERATING FACILITIES

The operating facilities are maintained and lubricated on an as-needed basis. The low level outlet is opened once a year for several hours for flushing.

### 4.4 DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT

No formal warning system is in effect. The operator reports emergency situations to his supervisor at the New Haven Water Company.

### 4.5 EVALUATION

Although informal, the operation and maintenance procedures are generally good, however, there are some areas requiring improvement. A formal program of operations and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.

## SECTION 5: HYDRAULIC/HYDROLOGIC

### 5.1 EVALUATION OF FEATURES

a. General - Glen Lake Dam is a low storage project primarily intended to "provide workable head conditions for the Sargent River portion of the West River System", according to the report entitled "Report on Flood Flows and Spillway Capacities, West River System Dams" by Malcolm Pirnie Engineers, dated January, 1967 and included in Appendix B.

b. Design Data - No hydraulic/hydrologic data was available for the original design of the dam. There is design data available for the 1967 spillway redesign. The above mentioned Malcolm Pirnie Engineers report dated January, 1967, was submitted after a report by Mr. Joseph W. Cone, Dam Consultant to the Water Resources Commission, dated June 26, 1965, indicated that the spillway capacity of Glen Lake Dam was inadequate, and that Glen Lake Dam would be overtopped by a storm larger than the October 1955 storm (See Appendix B).

Malcolm Pirnie Engineers considered a peak inflow due to a 1000-year storm and concluded the spillway should be lowered 2 feet for such a design storm, giving a total of 6 feet of freeboard to the top of the dam. A larger storm was considered for the redesign however, "in view of the uncertainty of future flood conditions." The Westfield, Massachusetts storm of August, 1955 was used, as it produced peak flows on the order of 50 percent larger than the 1000-year storm. Using the Westfield storm, it was concluded that the spillway should be lowered 5 feet, as it subsequently was.

c. Experience Data - No information on serious problem situations arising at the dam was found, and the dam has not been overtopped. The storm of October 1955 produced flows 2.7 feet above the spillway crest, which amounted to an available freeboard of 1.3 feet or a water surface elevation of 226.0 with the spillway crest at elevation 223.3 at that time.

d. Visual Observations - Under very high flows, the service bridge spanning the spillway could retain large floating debris resulting in an obstruction of the spillway. Debris in the form of wood was noted on the spillway crest and apron in minor amounts.

e. Test Flood Analysis - The test flood for this high hazard, intermediate size dam is equivalent to the Probable Maximum Flood (PMF). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge", dated March, 1978, peak inflow to the reservoir is 8,600 cfs (Appendix D-2); peak outflow is 8,220 cfs with the dam overtopped 1.6 feet (Appendix D-9). Based upon our hydraulics computations, the spillway capacity is 4,100 cfs, which is approximately 50% of the routed Test Flood outflow at the top of dam, elevation 227.3.

In computing peak inflows to Glen Lake due to the Probable Maximum Flood (PMF), it should be noted that a large portion of the Glen Lake watershed is regulated by Lake Chamberlain, an upstream reservoir of relatively large surface area and storage capacity. Based upon our computations, Lake Chamberlain, due to its storage effect, reduces the peak inflow to Glen Lake by approximately 1700 cfs for the PMF event.

f. Dam Failure Analysis - Utilizing the April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 61,600 cubic feet per second. A breach of the dam would result in a 27 foot depth of water immediately downstream of the dam at the residential structure. Inflow to Lake Dawson due to a breach of Glen Lake Dam would result in Lake Dawson Dam being overtopped by 0.9 feet, resulting in a 10,800 cfs outflow from Lake Dawson to the impact area around Konolds Pond about one mile further downstream. This rapid inflow to Konolds Pond would have potential for causing damage and loss of life at 5 to 10 residential structures along the shoreline of the pond.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - The visual inspection did not reveal any indications of stability problems.

b. Design and Construction Data - A stability analysis was performed by Malcolm Pirnie Engineers during the course of the 1967 study for increasing the spillway capacity which is included in Appendix B. This study concludes that the factor of safety against overturning, the critical consideration, is 1.18 with the reservoir water level at the top of the dam. The study further concludes that, as the actual uplift pressure is probably less than assumed, "we estimate that the dam is safe against overturning as long as the maximum water level does not exceed the top elevation of the existing non-overflow section." The study recommended no increase in the height of the dam.

There is not enough information available to perform a complete stability analysis of the dam, however the plans and existing information are sufficient to perform general stability calculations, such as those performed by Malcolm Pirnie Engineers.

c. Operating Records - The operating records do not include any indications of dam instability since its construction in 1906-1907 or since subsequent modifications of the concrete spillway weir in 1968-1969 were performed.

d. Post Construction Changes - The downstream face of the concrete gravity section was repaired using gunite in 1948.

A June 1965 inspection of the dam performed by Joseph W. Cone assumed that the left earth abutment is a dike with a corewall that is lower than the crest of the dam. This same report indicated that the earth section was nearly overtopped during the October 1955 flood.

In 1968-1969 the concrete weir was lowered 5 feet to increase the spillway capacity. This measure increased the structural stability of the dam by lowering the normal pool, thus reducing the possibility of high water conditions behind the dam.

e. Seismic Stability - The dam is in Seismic Zone 1, and according to the Recommended Guidelines, need not be evaluated for seismic stability.

## SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

### 7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and its past performance, the dam appears to be in good condition. No evidence of structural instability was observed in the dam or its appurtenances. There are some areas requiring attention, such as the concrete deterioration in the spillway and the gravity section, and the wet area downstream of the earth dike. Recommendations and remedial measures are presented in Section 7.2 and 7.3, respectively.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March, 1978, peak inflow to the reservoir is 8,600 cubic feet per second; peak outflow is 8,200 cubic feet per second, with the dam overtopped 1.6 feet. Based upon our hydraulic computations, the spillway capacity is 4,100 cubic feet per second, which is equivalent to approximately 50% of the routed Test Flood outflow.

b. Adequacy of Information - The information available includes evaluations of hydraulic capacity and structural design stability by Malcolm Pirnie Engineers. The assessment of the condition and stability of the dam was based, in part, upon these evaluations and on the visual inspection, past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within two years of the owner's receipt of this report.

d. Need for Additional Information - There is a need for more information as recommended in Section 7.2.

### 7.2 RECOMMENDATIONS

1. Based upon the rough computations in Appendix D, the spillway capacity will be exceeded by the Test Flood. A detailed evaluation of hydraulic/hydrologic computations performed by Malcolm Pirnie Engineers should be made by hydrologists/hydraulics engineers to determine if spillway modifications are warranted based upon Test Flood criteria utilized for this report. If deemed necessary by the evaluation above, more sophisticated flood routing should be undertaken by hydrologists/hydraulics engineers to refine the spillway design flood figures. A study should be undertaken to determine spillway capacity and overtopping potential. Recommendations should be made by the engineers and implemented by the owners to increase the project discharge based upon the refined spillway design flood figures.

## 7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time frame indicated in Section 7.1.c, and continued on a regular basis.

1. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation. The owner should develop a formal warning system with local officials for alerting downstream residents in case of an emergency.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. The New Haven Water Company has instituted a yearly program of technical inspection of all their dams, including Glen Lake Dam, by a consultant competent in the field of dam inspection. This program, in effect for 3 years, should be continued and should include the operation of all low level outlets.

4. The deteriorated concrete of the spillway and the downstream face of the dam should be repaired to prevent further concrete deterioration.

5. The seepage from the wet area downstream of the earth dike and left abutment should be monitored periodically in an attempt to evaluate its origin by observing changes in volume of the flow.

6. The cutting of grass, brush and trees, particularly those adjacent to the toe of the dam on the downstream earth slope of the left abutment, should be performed as part of the routine dam maintenance.

## 7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

# APPENDIX A

# INSPECTION CHECKLIST

## VISUAL INSPECTION CHECK LIST
## PARTY ORGANIZATION

PROJECT GLEN LAKE DAM

DATE: MAY 1, 1979

TIME: 12:50 PM

WEATHER: SUNNY, 70°F

W.S. ELEV. 218.5± U.S. ______ DN.S

| PARTY: | INITIALS: | DISCIPLINE: |
|---|---|---|
| 1. PETER M. HEYNEN | PMH | CAHN ENGINEERS, INC |
| 2. MIRON PETROVSKY | MP | CAHN ENGINEERS, INC. |
| 3. GEORGE STEPHENS | GS | CAHN ENGINEERS, INC. |
| 4. AL BUCHER | AB | NEW HAVEN WATER Co. |
| 5. CALVIN GOLDSMITH | C.G | CAHN ENGINEERS, INC |
| 6. | | |

| PROJECT FEATURE | INSPECTED BY | REMARKS |
|---|---|---|
| 1. CONCRETE GRAVITY DAM | PMH, MP, GS, AB | |
| 2. EARTH DIKE | PMH, MP, GS, AB | |
| 3. CONCRETE SPILLWAY | PMH, MP, GS | |
| 4. CONCRETE GATE HOUSE | PMH, GS, AB | |
| 5. OUTLET MASONRY HEADWALL | PMH, MP | |
| 6. | | |
| 7. | | |
| 8. | | |
| 9. | | |
| 10. | | |
| 11. | | |
| 12. | | |

PERIODIC INSPECTION CHECK LIST

Page A-2

PROJECT GLEN LAKE DAM DATE MAY 1, 1979

PROJECT FEATURE CONCRETE GRAVITY DAM BY PMH, MP, GS, AB

| AREA EVALUATED | | CONDITION |
|---|---|---|
| DAM EMBANKMENT | | |
| Crest Elevation | | 227.3 |
| Current Pool Elevation | | 218.5 ± |
| Maximum Impoundment to Date | | NOT KNOWN |
| Surface Cracks | | NONE OBSERVED |
| Pavement Condition | | N/A |
| Movement or Settlement of Crest | | NONE OBSERVED |
| Lateral Movement | | |
| Vertical Alignment | | APPEARS GOOD |
| Horizontal Alignment | | |
| Condition at Abutment and at Concrete Structures | | LEFT ABUTMENT IS WOODED AREA |
| Indications of Movement of Structural Items on Slopes | | N/A |
| Trespassing on Slopes | | |
| Sloughing or Erosion of Slopes or Abutments | | SOME EROSION OF D/S SLOPE : CRACKING & SPALLING NEAR JOINTS |
| Rock Slope Protection-Riprap Failures | | N/A |
| Unusual Movement or Cracking at or Near Toes | | NONE OBSERVED |
| Unusual Embankment or Downstream Seepage | | |
| Piping or Boils | | |
| Foundation Drainage Features | | N/A |
| Toe Drains | | |
| Instrumentation System | | |

## PERIODIC INSPECTION CHECK LIST



PROJECT GLEN LAKE DAM DATE MAY 1, 1979

PROJECT FEATURE EARTH DIKE BY PMH, MP, GS, AB

| AREA EVALUATED | | CONDITION |
|---|---|---|
| DIKE EMBANKMENT | | |
| Crest Elevation | | 227.3 $\pm$ |
| Current Pool Elevation | | 218.5 $\pm$ |
| Maximum Impoundment to Date | | NOT KNOWN |
| Surface Cracks | | NONE OBSERVED |
| Pavement Condition | | N/A |
| Movement or Settlement of Crest | | NONE OBSERVED |
| Lateral Movement | | |
| Vertical Alignment | | APPEARS GOOD |
| Horizontal Alignment | | |
| Condition at Abutment and at Concrete Structures | | ABUTMENTS ARE WOODED AREA |
| Indications of Movement of Structural Items on Slopes | | NONE OBSERVED |
| Sloughing or Erosion of Slopes or Abutments | | |
| Rock Slope Protection-Riprap Failures | | N/A |
| Unusual Movement or Cracking at or Near Toes | | NONE OBSERVED |
| Unusual Embankment or Downstream Seepage | | BELOW D/S STONE WALL IS WET AND SPRING AREA |
| Piping or Boils | | NONE OBSERVED |
| Foundation Drainage Features | | NOT KNOWN |
| Toe Drains | | |
| Instrumentation System | | N/A |
| Trespassing on Slopes | | NONE OBSERVED |

## PERIODIC INSPECTION CHECK LIST

Page A-4

PROJECT GLEN LAKE DAM                    DATE MAY 1, 1979

PROJECT FEATURE CONCRETE GATE HOUSE      BY PMH, GS, AB

| AREA EVALUATED | | CONDITION |
|---|---|---|
| OUTLET WORKS-CONTROL TOWER | | |
| a, Concrete and Structural | | |
| General Condition | | GOOD |
| Condition of Joints | | NOT OBSERVED |
| Spalling | | NONE OBSERVED |
| Visible Reinforcing | | |
| Rusting or Staining of Concrete | | |
| Any Seepage or Efflorescence | | |
| Joint Alignment | | NOT OBSERVED |
| Unusual Seepage or Leaks in Gate Chamber | | |
| Cracks | | NONE OBSERVED |
| Rusting or Corrosion of Steel | | |
| b) Mechanical and Electrical | | |
| Air Vents | | N/A |
| Float Wells | | |
| Crane Hoist | | |
| Elevator | | |
| Hydraulic System | | |
| Service Gates | | ALL GATES ARE OPERABLE |
| Emergency Gates | | N/A |
| Lightning Protection System | | |
| Emergency Power System | | |
| Wiring and Lighting System | | |

PERIODIC INSPECTION CHECK LIST

Page A-5

PROJECT GLEN LAKE DAM DATE MAY

PROJECT FEATURE OUTLET MASONRY HEADWALL BY PMH, MP

| AREA EVALUATED | | CONDITION |
|---|---|---|
| OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL | | |
| General Condition of ~~Concrete~~ MASONRY | | FAIR |
| Rust or Staining | | NONE OBSERVED |
| Spalling | | NONE OBSERVED |
| Erosion or Cavitation | | NONE OBSERVED |
| Visible Reinforcing | | N/A |
| Any Seepage or Efflorescence | | NONE OBSERVED |
| Condition at Joints | | SOME OPEN |
| Drain Holes | | N/A |
| Channel | | |
| Loose Rock or Trees Overhanging Channel | | NONE OBSERVED |
| Condition of Discharge Channel | | GRAVEL & STONE STREAMBED |

## PERIODIC INSPECTION CHECK LIST



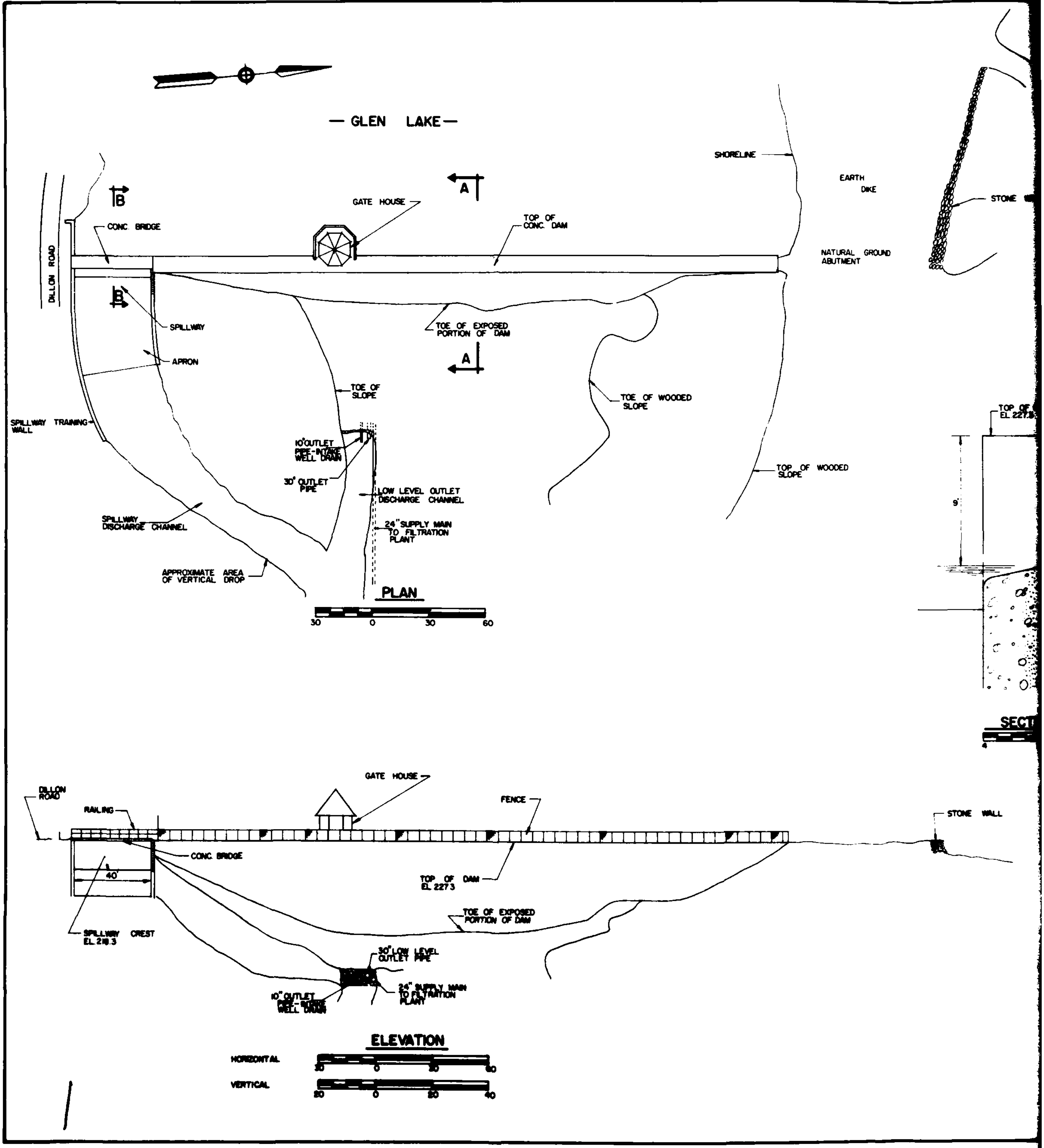
PROJECT GLEN LAKE DAM DATE MAY 1, 1979

PROJECT FEATURE CONCRETE SPILLWAY BY PMH, MP, GS

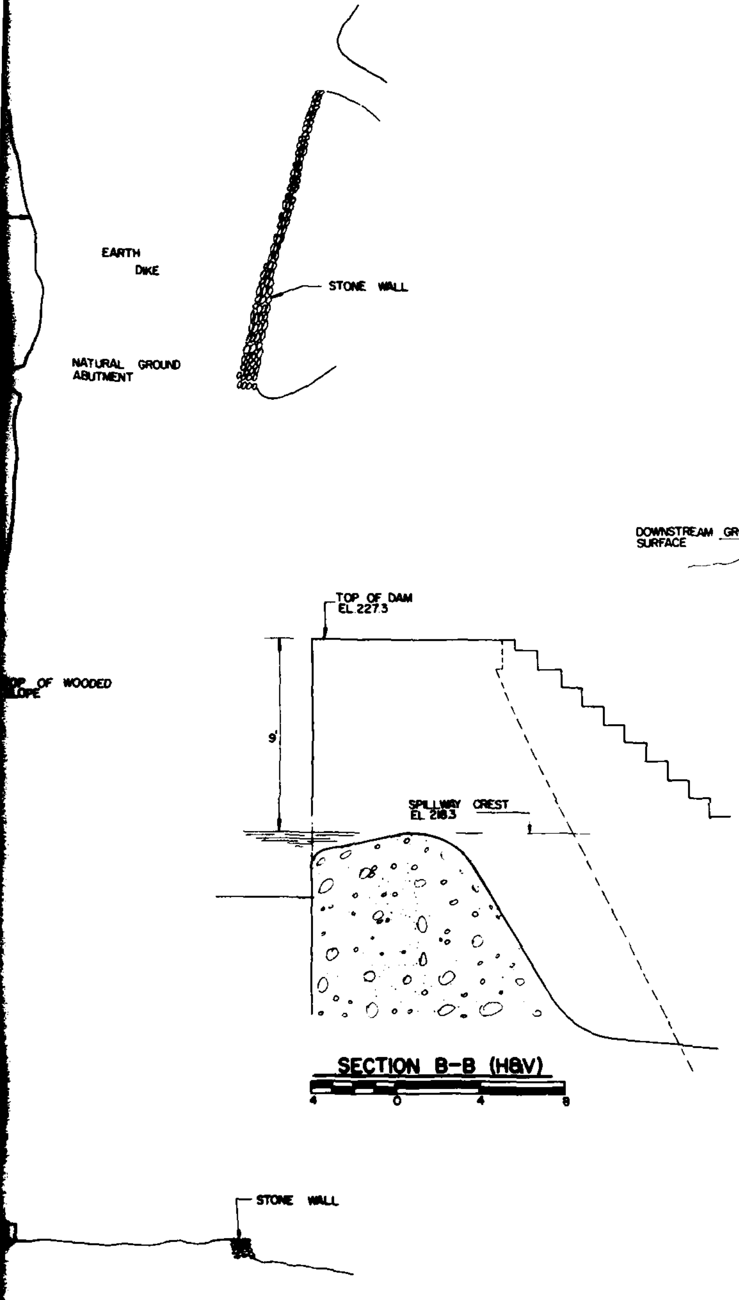
| AREA EVALUATED | | CONDITION |
|---|---|---|
| OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS | | |
| a) Approach Channel | | N/A |
| General Condition | | |
| Loose Rock Overhanging Channel | | |
| Trees Overhanging Channel | | |
| Floor of Approach Channel | | |
| b) Weir and Training Walls | | |
| General Condition of Concrete | | GOOD |
| Rust or Staining | | NONE OBSERVED |
| Spalling | | SOME CRACKING & SPALLING OF RETAINING WALLS |
| Any Visible Reinforcing | | NONE OBSERVED |
| Any Seepage or Efflorescence | | SOME EFFLORESCENCE NEAR CRACKS |
| Drain Holes | | N/A |
| c) Discharge Channel | | |
| General Condition | | GOOD |
| Loose Rock Overhanging Channel | | NONE OBSERVED |
| Trees Overhanging Channel | | ON RIGHT BANK |
| Floor of Channel | | BOULDER AND ROCK |
| Other Obstructions | | DEBRIS OF TREES IN SPILLWAY AND SPILLWAY CHANNEL |

# APPENDIX B

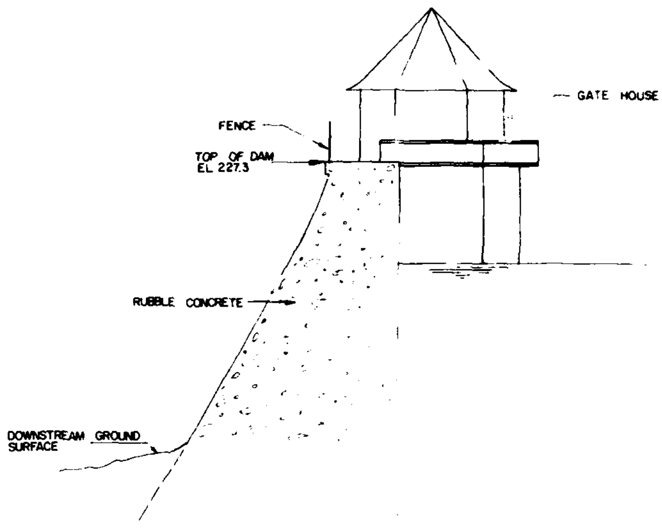
# ENGINEERING DATA AND CORRESPONDENCE



— GLEN LAKE —
DILLON ROAD
CONC. BRIDGE
GATE HOUSE
TOP OF CONC. DAM
SHORELINE
EARTH DIKE
STONE
NATURAL GROUND ABUTMENT
SPILLWAY
APRON
TOE OF EXPOSED PORTION OF DAM
TOE OF SLOPE
TOE OF WOODED SLOPE
SPILLWAY TRAINING WALL
10" OUTLET PIPE-INTAKE WELL DRAIN
30" OUTLET PIPE
LOW LEVEL OUTLET DISCHARGE CHANNEL
TOP OF WOODED SLOPE
SPILLWAY DISCHARGE CHANNEL
24" SUPPLY MAIN TO FILTRATION PLANT
APPROXIMATE AREA OF VERTICAL DROP
PLAN
30 0 30 60
9'
SECT
DILLON ROAD
RAILING
GATE HOUSE
FENCE
STONE WALL
CONC. BRIDGE
40'
TOP OF DAM EL. 227.3
TOE OF EXPOSED PORTION OF DAM
SPILLWAY CREST EL. 218.3
30" LOW LEVEL OUTLET PIPE
24" SUPPLY MAIN TO FILTRATION PLANT
10" OUTLET PIPE-INTAKE WELL DRAIN
ELEVATION
HORIZONTAL
30 0 30 60
VERTICAL
20 0 20 40




SECTION B-B (H&V)




SECTION A-A(H&V)

## NOTES

1 THIS PLAN WAS COMPILED FROM A PLAN OF INCOMPLETE SPOT ELEVATIONS ENTITLED "NEW HAVEN WATER COMPANY, ELEVATIONS IN VICINITY OF GLEN DAM" BY CLARENCE BLAIR ASSOCIATES, DATED SEP 1966, AND FROM PLANS ENTITLED "NEW HAVEN WATER CO WOODBRIDGE GLEN DAM" BY ALBERT B HILL DATED MARCH 1906 AND UNTITLED PLAN BLAIR & MARCHANT, INC DATED SEP 1934 AND CAHN SUPPLEMENTARY FIELD SURVEY NOT ALL TOPOGRAPHIC AND/OR STRUCTURAL FEATURES ARE IDENTIFIED

2. ELEVATIONS SHOW ARE MEAN SEA LEVEL, CONVERTED FROM MEAN HIGH WATER ELEVATIONS SHOWN ON EXISTING PLANS
MHW+3.3=MSL

| | | | |
|---|---|---|---|
| CAHN ENGINEERS INC.<br>WALLINGFORD, CONNECTICUT<br>ENGINEER | | U.S. ARMY ENGINEER DIV NEW ENGLAND<br>CORP OF ENGINEERS<br>WALTHAM, MASS | |
| NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS<br>PLAN, ELEVATION AND SECTIONS<br>**GLEN LAKE DAM**<br>SARGENT RIVER — WOODBRIDGE, CONNECTICUT | | | |
| DRAWN BY | CHECKED BY | APPROVED BY | SCALE AS NOTED |
| M N | CBS | PMH | DATE JUNE 1979 — SHEET B-1 |

2

## LIST OF EXISTING PLANS

"New Haven Water Co.
Contour Map, Glen Lake
Town of Woodbridge, Ct."
Office of Albert B. Hill, Consulting Engineer
Feb., 1905

"New Haven Water Co.
Woodbridge Glen Dam"
Office of Albert B. Hill, Consulting Engineer
March, 1906

"New Haven Water Co.
Section of Woodbridge Glen Dam"
Office of Albert B. Hill, Consulting Engineer
March, 1906

"New Haven Water Co.
Plans for Gatehouse
Woodbridge Glen Reservoir"
Office of Albert B. Hill, Consulting Engineer
March, 1906

"New Haven Water Co.
Gatehouse Inlets
Woodbridge Glen Dam"
Office of Albert B. Hill, Consulting Engineer
Sept., 1906

"New Haven Water Co.
West River System
Alterations to Glen Lake Gatehouse"
Blair & Marchant, Inc.
Sept., 1934

"New Haven Water Co.
Elevations in Vicinity of Glen Dam
Woodbridge, Conn."
Clarence Blair Assc., Inc.
Sept. 1966

"New Haven Water Co.
Cross Section in Vicinity of Glen Dam
Woodbridge, Conn."
Clarence Blair Assc., Inc.
Feb., 1967

## SUMMARY OF DATA AND CORRESPONDENCE

| DATE | TO | FROM | SUBJECT | PAGE |
|---|---|---|---|---|
| May 19, 1964 | Files | Water Resources Commission, Supervision of Dams | Inventory Data | B-4 |
| Aug. 12, 1974 | Files | New Haven Water Co. | Statistics on dams | B-5 |
| Apr. 29, 1963 | A.L. Corbin, President, New Haven Water Co. | Joseph A. Novaro, Chief Engineer, New Haven Water Co. | Hydraulic data and computations on West River Watershed | B-8 |
| Apr. 12, 1965 | Joseph W. Cone, P.E. | Joseph A. Novaro | Additional hydraulic data on West River watershed | B-11 |
| June 26, 1965 | William P. Sander Water Resources Commission | Joseph W. Cone, P.E. | Summary of report concerning dams owned by the New Haven Water Co. on the West and Sargent Rivers | B-12 |
| July 15, 1966 | William Wise, Director Water Resources Commission | Joseph A. Novaro | Progress report on studies on West River System | B-18 |
| Jan., 1967 | New Haven Water Co. | Malcolm Pirnie Engineers | Excerpts from report on flood flows and spillway capacities on West River System with recommendations to increase spillway capacities | B-19 |
| Aug. 2, 1967 | New Haven Water Co. | Malcolm Pirnie Engineers | Report on effects of maximum possible storm on spillways of West River system | B-41 |

| DATE | TO | FROM | SUBJECT | PAGE |
|---|---|---|---|---|
| Dec., 1967 | New Haven Water Co. | Malcolm Pirnie Engineers | Plans for modifications to increase spillway capacities, excerpted from contract documents | B-44 |
| July 26, 1968 | Water Resources Commission | Joseph A. Novaro, Chief Engineer, New Haven Water Co. | Application for construction permit for spillway modifications | B-47 |
| Aug. 20, 1968 | Files | William H. O'Brien, III, Water Resources Commission | Additional data from Malcolm Pirnie on new spillway capacity | B-49 |
| Aug. 19, 1969 | Water Resources Commission | Joseph A. Novaro | Notification of completion of spillway lowering, with record drawings | B-50 |
| Mar. 1, 1971 | Files | William H. O'Brien, III | Notation of trees growing near downstream toe of dam | B-54 |
| June 29, 1973 | Richard P. McHugh, New Haven Water Co. | Victor F. Galgowski, Supt. of Dam Maintenance Water and Related Resources, Dept. of Environmental Protection | Suggestion for removal of trees | B-55 |
| June 29, 1973 | Richard P. McHugh | Dan W. Lufkin, Commissioner, Water and Related Resources | Certificate of approval of spillway modifications | B-56 |

No. ~~WS 14~~

**WATER RESOURCES COMMISSION**
**SUPERVISION OF DAMS**
**INVENTORY DATA**

Long 72-58.7
Lat. 41-22.6

Inventoried By WS

Date 19 MAY 1964

Name of Dam or Pond GLEN DAM RESERVOIR

Code No. WS 3.6 SG 0.2

Nearest Street Location DILLON ROAD

Town WOODBRIDGE

U.S.G.S. Quad. MOUNT CARMEL

Name of Stream SARGENT RIVER

Owner NEW HAVEN WATER COMPANY

ok 1/73

Address 100 Crown St.
New Haven, Conn.

---

Pond Used For WATER SUPPLY

Dimensions of Pond: Width 300 FEET Length 2500 FEET Area ~~18~~ 26 ACRES

Total Length of Dam ~~300~~ 370 FEET Length of Spillway ~~30 FEET~~ 40

Location of Spillway SOUTH END OF DAM

Height of Pond Above Stream Bed ~~60~~ 46 FEET

Height of Embankment Above Spillway 4 FEET

Type of Spillway Construction CONCRETE

Type of Dike Construction CONCRETE

Downstream Conditions ROUTE 69

1:07 Summary of File Data

Remarks

B-4

Would Failure Cause Damage? YES Class B

NEW HAVEN WATER COMPANY

STATISTICS ON DAMS*

NAME Glen

SUPPLY SYSTEM West River

LOCATION Woodbridge

DATES: ORIGINAL CONSTRUCTION 1906-1907

ADDITIONS, ALTERATIONS 1968-1969

| | MEAN HIGH WATER ELEVATION | LENGTH |
|---|---|---|
| CREST** | 224 | 390 Ft. |
| TOP OF CORE WALL | | |
| SPILLWAY | 215 | |
| B. O. AXIS | 177± | |
| BED OF RIVER | 162± | |
| DEEPEST FOUNDATION | 149± | |

FREEBOARD: CREST TO SPILLWAY 9 Ft.

CREST TO TOP OF CORE WALL

HEIGHT: CREST TO BED OF BROOK 62± Ft.

CREST TO DEEPEST FOUNDATION 75± Ft.

TYPE Concrete Gravity Section

TOP WIDTH--MAX. BOTTOM WIDTH (Ft.) 9 -- 48

UPSTREAM SLOPE H/V Vertical

DOWNSTREAM SLOPE H/V 6/10

TRIBUTARY WATERSHED (Square Miles) 5.6

RESERVOIR AREA (Acres) 23.0

RESERVOIR TOTAL STORAGE (MG) 157

RESERVOIR USABLE STORAGE (MG) 153

*See individual sheets for more details
**Crest Length includes spillway

Date 8/12/74

NEW HAVEN WATER COMPANY



DATE Aug. 1974

NAME OF DAM GLEN

TYPE

Concrete, gravity section, Large stones placed in the concrete. Spillway located on the south end of the dam

LOCATION

In the Town of Woodbridge on Sargent River, on north side of Dillon Road and approximately 800 feet west of State Highway No. 69 known as the Litchfield Turnpike

SUPPLY SYSTEM West River

DATE OF CONSTRUCTION

ORIGINAL 1906 - 1907

OTHER 1968-1969 - Spillway lowered 5 feet to Elevation 215 in order to provide increase of spillway capacity for flood flows. The entrance channel to the spillway was widened and deepened also.

1948 - Gunite repairs to dam and spillway surfaces.

| ENGINEER | 7. CONTRACTOR |
|---|---|
| 1906-1907 Albert B. Hill | New York Continental Jewell Filtration Company and Upson & Grannis, Contractors |
| 1968-9 Malcolm Pirnie Engineers | C.W. Blakeslee & Sons, Inc. |
| 1948 Clarence Blair Assoc. Inc. | Cement Gun Company |

| | ELEVATION | | LENGTH (Feet) | MISC. |
|---|---|---|---|---|
| CREST | 224 | M.H.W. | 390' | Length includes spillway |
| SPILLWAY | 215 | M.H.W. | 40 | Ogee type |
| AXIS OF B.O | ± 177 | " | | 30" C.I. Pipe |
| BED OF RIVER | ± 162 | " | | |
| DEEPEST FNDN | ± 149 | " | | Rock |

NEW HAVEN WATER COMPANY Pg. 2

GLEN DAM DATE ______

3 HEIGHT FROM BED OF BROOK ± 62 Feet

4 HEIGHT FROM DEEPEST FOUNDATION ± 75 Feet

5. TOP WIDTH Includes coping over-hang on downstream side 9 Feet

6 MAXIMUM WIDTH AT BOTTOM 48 Feet

17. UPSTREAM SLOPE Vertical

18 DOWNSTREAM SLOPE 6 Hor. on 10 Ver.

19. FREE BOARD - SPILLWAY TO CREST 9 Feet

- SPILLWAY TO TOP OF COREWALL ______ Feet

21. MISC. DATA

Dam is founded on rock - a tight phyllite

2 WATERSHED TRIBUTARY TO:

UPSTREAM DAMS Chamberlain Dam 3.9 Sq. Mi

THIS DAM 1.7 Sq. Mi.

TOTAL WATERSHED TRIBUTARY TO THIS DAM 5.6 Sq. Mi

22. RESERVOIR AREA AT FLOW LINE 23.0 Acres

23. RESERVOIR CAPACITY AT FLOW LINE 157 Mil. G.

24. RESERVOIR USABLE CAPACITY (To lowest outlet) 153 Mil. Ga

25. UPSTREAM DAMS

Chamberlain Dam of N.H. Water Co

26 DOWNSTREAM DAMS

Lake Dawson Dam of N.H. Water Co

Konold Pond

Pond Lilly Dam of The Pond Lilly Co.

April 29, 1963.

To: Mr. A. L. Corbin, Jr., President
From: Joseph A. Novaro, Chief Engineer

Re: West River Watershed.

Flood conditions in 1955 at and upstream from the Whalley Avenue bridge in Westville, generally attributed to the West River, actually were the result of heavy storm runoffs from several watersheds:

1. West River lying west and south of West Rock, eventually passing under the Whalley Avenue bridge.

2. An area starting at the Yale Golf Course ponds and extending north to the Fountain Street - Whalley Avenue area, draining to West River.

3. Wintergreen Brook lying east of West Rock. It enters West River about 600 feet north of the Whalley Avenue bridge.

4. Farm Brook, east of West Rock, starting about one mile north of Paradise Park in Hamden and draining south into Wintergreen brook about 1900 feet southeast of the Springside Home.

5. An unnamed brook lying between 3 and 4 above, which starts about one-half mile west of Paradise Park in Hamden and drains south into Wintergreen Brook at a point in the Brookside Housing area of New Haven.

6. Beaver Pond watershed which stretches approximately from Arch Street in Hamden south to Goffe Street in New Haven. The brook from Beaver Pond runs southwest, entering Wintergreen Brook about 900 feet north of the Whalley Avenue bridge.

The watersheds tributary to the Whalley Avenue bridge total 29.3 square miles which I have broken down, for analysis, into three main areas:

| | | |
|---|---|---|
| North of and tributary to Dawson Dam | | 13.9 sq. mi. |
| " " " " " Wintergreen Dam | | 1.5 sq. mi. |
| Remaining watershed | | 13.9 sq. mi. |
| | Total | 29.3 sq. mi. |

The New Haven Water Company owns approximately one square mile of the 1.5 square miles of watershed tributary to Lake Wintergreen and about 8 square miles of the 13.9 square miles of watershed tributary to Lake Dawson. The balance is owned by others. The Company owned land, used for water supply purposes only, and well forested, has not contributed to any increase in flood runoff. In fact the Company's forestry program has effected some decrease in the rate of storm water runoff from the land.

The balance of the land owned by others and draining to the Whalley Avenue bridge has been and will continue to be developed for housing, schools, industry and colleges. Their roofs, driveways, streets and parking areas increase the amount and rate of storm water runoffs and storm water sewers, where installed, accelerate the runoff.

Our reservoirs generally start to go down early in June and continue to go down until late in the year. About half the year's refilling starts about the middle of November and about the middle of December the rest of the years. Occasionally, as recently experienced, our reservoirs start to refill in January and very occasionally in February. Our reservoirs thus are in a position during the hurricane season to receive and retain a large portion, and sometimes all, of the storm runoff from the 15.4 square miles tributary to them.

In August 1955 hurricane Connie, followed by Dianne, brought heavy rains to this area. Dianne caused considerable damage in Milford and the lower Naugatuck Valley. In the period August 8 to 14 inclusive rainfall at Lake Dawson totalled 4.14". On August 18 and 19 hurricane Dianne brought an additional 6.67". In one 24 hour period 4.87" fell at Dawson.

In this extended storm period our reservoirs received and retained 477 million gallons of water. Glen, Watrous, Chamberlain, Bethany and Wintergreen retained all the runoff reaching them, allowing nothing to go downstream. Dawson, on August 19th, with its small tributary watershed of 0.8 square miles, finally filled but the depth of flow over the spillway was only one half an inch. The data is listed herewith:

| | Before the Storms | | After the Storms | |
|---|---|---|---|---|
| | Reservoir Level | Million gals. to fill | Reservoir Level | Million gals. to fill |
| Dawson | down 0' 1/2" | 1 | Full | 0 |
| Glen | " 21' 3" | 140 | down 10' 3" | 81 |
| Watrous | " 6' 4" | 209 | " 2' 7" | 86 |
| Chamberlain | Empty | 164 | " 4' 10" | 49 |
| Bethany | down 4' 6" | 138 | Full | 0 |
| Wintergreen | " 6' 6" | 66 | down 2' 5" | 25 |
| | Totals | 718 | | 241 |

Amount retained 718 - 241 = 477 million gallons.
In addition about 9 million gallons per day throughout the entire storm period was also utilized for water supply purposes.

The heavy storm on October 14 to 17 inclusive in 1955 produced floods and considerable damage in the Westville area. Our rain gauge at Dawson registered 8.84" of rainfall in this period. Of this 5.85" fell in one 24 hour period alone. Our reservoirs were all full after this storm but prior to filling they stored and retained 251 million gallons of water as shown in the data below:

| | Reservoir level | Million gals. to fill |
|---|---|---|
| Dawson | over 0' 1/2" | 0 |
| Glen | down 5' 1" | 45 |
| Watrous | " 2' 6" | 83 |
| Chamberlain | " 9' 4" | 88 |
| Bethany | " 0' 1/2" | 2 |
| Wintergreen | " 3' 3" | 33 |
| | | 251 |

In addition, at the height of the storm water runoff our reservoirs temporarily stored 215 million gallons additional above their spillways, preventing even higher flood levels down stream by releasing this over a greater period of time. The data is herewith:

| | Depth above Spillway | Surface Acres | Acre-feet | Million gals. |
|---|---|---|---|---|
| Dawson | 2' 5" | [illegible] | 172 | 55.5 |
| Glen | 2' 7" | 27 | 70 | 22.6 |
| Watrous | 1' 11" | 110 | 211 | 68.1 |
| Chamberlain | 2' 0" | 37 | 74 | 23.9 |
| Bethany | 0' 11" | 106 | 97 | 31.3 |
| Wintergreen | 1' 0" | 44 | 44 | 14.1 |
| | | | | 215.5 |

The effect of reservoir storage above the spillway level on downstream flood conditions can be checked by comparing the flood runoff from the reservoir-controlled watersheds with that of the other watersheds as follows:

1. From lake level records (depths on spillway) I have computed that at peak runoff approximately 1425 cubic feet per second were passing our Dawson and Wintergreen dams. For the 15.4 sq. miles of tributary watershed this is an average runoff rate of 92 cubic feet per second per square mile of watershed. (September 1938 hurricane runoffs were in the 40 to 80 range).

2. The peak flow under the Whalley Avenue bridge, computed by Consultants for the State, was 3,525 cubic feet per second. Subtracting above 1425 cusec. leaves 2,100 cusec. contributed by the remaining, uncontrolled 13.9 square miles or at an average runoff rate of 151 cubic feet per sec. per square mile.

Peak runoff rate from the uncontrolled portions of the watersheds therefore was about 50 per cent higher than from the controlled watersheds for this particular storm. This is not surprising when you consider the absence of large reservoirs and the large amount of impervious surfaces in the built up residential, commercial, school and industrial areas.

Consultants for the State reported that a 48 inch diameter sewer suspended under the floor of the bridge restricted the flow area of the bridge, accentuating flood conditions upstream. In order to pass the computed possible flood flow at this point - larger than the 3,525 cusec. - the Consultants recommended that the sewer be replaced with a siphon under West River and that additional waterway capacity be provided by widening the bridge.

Since this flood New Haven Water Company has raised Chamberlain Dam 35 feet increasing its storage from 164 million gallons to 894 million gallons. Thus in the fugure additional space has been provided to store and retain flood runoffs.

While Company owned land will remain well forested, retaining normal yield and runoff, the areas owned by others will continue to be developed for other uses - uses which will inevitably increase the amount of storm runoff and the rate of runoff.

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT 06506

Tel. MA 4-9803

April 12, 1965

Mr. Joseph W. Cone,
Civil Engineer,
124 Havemeyer Place,
Greenwich, Conn.

Dear Mr. Cone:

Referring to your letter of April 2, 1965, we enclose the following:

1. Data forms for Chamberlain, Glen, Bethany, Watrous and Dawson Dams.

2. Plans for above dams.

3. Sanitation map showing limits of watershed tributary to above dams.

In the period from 1937 to the present, depths over the spillways of the above dams in most cases have been less than one foot.

Our rain gauge at Lake Dawson recorded a total of 4.14" in the August 8 - 14, 1955 storm. It recorded 6.67" on August 18-19, 1955. In one 24-hour period rainfall totalled 4.87". None of the runoff went downstream but Lake Dawson was full at the end of the storm.

The Lake Dawson rain gauge recorded 8.84" of rain in the October 14-17, 1955 storm, of which 5.85" fell in one 24-hour period. This storm filled the four upstream reservoirs. Maximum depths on spillways occurred on October 16, 1955 and are recorded on the data forms.

Chamberlain Dam was raised in 1958-1959 and a new larger spillway was provided. Storage was increased from the original 164 million gallons to the present 894 million gallons.

If you will let me know when you wish to make a field inspection, I will be glad to make the necessary arrangements.

Yours very truly,
NEW HAVEN WATER COMPANY

Joseph A. Novaro

Joseph A. Novaro
Chief Engineer

RECEIV

1965

REPORT

CONCERNING DAMS

Owned by

NEW HAVEN WATER CO.

BETHANY

WATROUS

CHAMBERLAIN

GLEN

DAWSON

on the

WEST & SARGENT RIVERS

J. W. Cone P.E.
June 1965

# INDEX

## Part I



## Part II

NOTE: Maps, graphs, etc., are in separate folder.

York License 4788
Connecticut Registration 4

Joseph W. Cone
Civil Engineer
124 Havemeyer Place
Greenwich, Connecticut
06830

Telephone
Townsend 9-2152

June 26, 1965

Mr. William P. Sander
Water Resources Commission
State Office Building
Hartford 15, Conn.

Re: Dams #35 - 1 to 5
New Haven Water Co.

Dear Mr. Sander:

First, I apologize for not completing this assignment more promptly; reasons being that a low quality virus for over a month left me with no pep mentally or physically, and delays in obtaining certain plans and information.

The assignment was- "we would like to know the present condition of these dams" - Bethany - Watrous - Dawson on West River and Chamberlain - Glen on Sargent River, a tributory to West River above Dawson Dam.

In my opinion, the "condition" of these dams is good as regards masonry of the three masonry gravity dams and the upkeep of two earth embankment dams.

But as regard to whether or not the dams are safe, particularly as regard spillway capacity, my opinion is as follows:

35-1 Bethany Spillway is inadequate. However a thin sheet over a length of 990' will do comparatively little damage except to highway. The gravity section is safe.

35-2 **Watrous** Generally same remarks as for Bethany.

35-3 **Chamberlain** Spillway is adequate in every respect as is the dam. It is reassuring to find a spillway that will carry 1525 cfs per sq. mi. on 4.1 sq. mi. Note Items #26 & 28 on Data Sheet.

35-4 **Glen** Spillway is nowhere near adequate. In fact, Oct. '55 flood nearly overtopped earth section at left or east abutment. Section of dam is safe.

Right abutment should be raised to protect highway.

Left abutment should be investigated:-

(a) To determine whether or not there is a core wall.

(b) Possibility of emergency spillway or fuse plug.

(c) Note Items #26 & 28 on Data Sheet.

35-5 **Dawson** Present spillway is entirely inadequate to carry probable floods of the present and future. In fact, the dam would have been overtopped if certain saving factors had not been present in Oct. 1955.

(a) Not an excessive rainfall, only about R of 50 yr. (Compare with precipitation graphs)

(b) Several of reservoirs were below FL (See data notes by Navaro which you have)

(c) Flood Q '55 at Dawson of about 2100 cfs has an R value 3.8 (2100 ÷ 560) equivalent to 120 yr on old Conn. curve and 55 yr on revised 1965 curve. (See graph PL 13)

Items #26 & 28 on Data Sheet are particularly illuminating.

It does not need a lively imagination to visualize what would happen to Westville and New Haven if Dawson should be overtopped; Norwich failure would be peanuts comparatively.

A brief discussion of pertinent data and situations follows. Also there are prints of sections of dams, precipitation graphs and various other graphs that I used or are pertinent to this investigation for general information or checking purposes.

Please excuse the informality and crudness of the matter submitted, the objective being to reduce costs to the minimum.

I would observe that Mr. Navaro, Mr. Ferris and Mr. Reynolds of the New Haven Water Co. were most cooperative as was Mr. Thomas of the U.S. Geological Survey.

My recommendation is that the New Haven Water Co. be advised that their consulting engineers should investigate the entire system, with particular emphasis on conditions at Glen and Dawson, and submit corrective measures.

Yours very truly,

J. W. Cone

JWC/dr

Enc: Part II
Photos (11)

**NEW HAVEN WATER COMPANY**
**NEW HAVEN, CONNECTICUT 06506**

July 15, 1966

STATE WATER RESOURCES COMMISSION
RECEIVED
1 1966
ANSWERED
REFERRED
FILED

Mr. William Wise, Director,
Water Resources Commission,
State Office Building,
Hartford 15, Conn.

Dear Mr. Wise:

As promised we are writing to report progress to date on the studies of our West River System.

Our consultants, Malcolm Pirnie Engineers, have gathered all available data concerning the 1955 hurricane storms and the characteristics of the West River and Sargent River watersheds, reservoirs, and dams. This information has been supplemented by a field investigation by them.

They are using the unit hydrograph method of analysis. Their first step is to reconstruct one of the 1955 storms and route it through the watersheds. If, by this procedure, they can produce, within reason, the conditions which were observed at the various dams during the 1955 storms, the characteristics of the unit hydrograph and the procedure can be considered verified.

With the procedure verified, they plan to route a 100-year storm and a 1000-year storm through the reservoir systems. The results of these runs will be used to determine what improvements to recommend. Stability analyses will be made after the design hydraulic conditions have been determined.

To date our consultants have completed their general hydrologic investigations; have constructed unit-hydrographs to be used with the drainage areas tributary to each dam and reservoir; have selected and arranged rainfall data to be used for the 1955 storm and for the 100-year and 1000-year storms and have computed in-flow hydrographs into each of the reservoirs for the 1955 storms. Rating curves are being computed for each spillway. When these computations are completed the 1955 storm will be routed through the system in order to verify the procedure.

Our consultants advise that their final report should be ready by the end of September.

Yours very truly,
NEW HAVEN WATER COMPANY

Joseph A. Novaro
Joseph A. Novaro
Chief Engineer

772 2550

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT

REPORT ON
FLOOD FLOWS AND SPILLWAY CAPACITIES
WEST RIVER SYSTEM DAMS

JANUARY 1967

MALCOLM PIRNIE ENGINEERS
Office Park
226 Westchester Avenue
White Plains, New York 10604

# I. PURPOSE AND SCOPE

On June 26, 1965, Mr. Joseph W. Cone, Dam Consultant to the Water Resources Commission, reported to the Commission the results of an assignment by the Commission to study the present condition of the dams owned by the New Haven Water Company on the West River and its tributaries. Mr. Cone's report, which will be summarized later, was not intended to be a comprehensive study of the dams in question. It indicated that spillway capacities on four of the five dams concerned were less than considered desirable, and recommended that a more detailed engineering study be made by the Company to determine deficiencies, if any, and the necessary corrective measures.

Subsequently, Malcolm Pirnie Engineers was authorized to study the adequacy of all spillways in the West River system and make recommendations as to changes and additions.

## II. DAMS INVESTIGATED

The dams under investigation store water for the West River or Woodbridge system and are located on the West and Sargent Rivers of Connecticut. The dams impound runoff from a total drainage area of 13.6 square miles, the southern extremity of which lies approximately one and four-tenths miles north of the New Haven city line. The system has a yield of about 10 million gallons per day.

The following tabulation contains pertinent data concerning the dams and reservoirs studied.

| | Bethany | Watrous | Chamberlain | Glen | Dawson |
|---|---|---|---|---|---|
| Date Built | 1892-1931 | 1914 | 1899-1959 | 1907 | 1889 |
| Drainage Area S.M. | | | | | |
| Direct* | 3.8 | 3.3 | 4.0 | 1.7 | 0.8 |
| Total | 3.8 | 7.1 | 4.0 | 5.7 | 13.6 |
| Res. Cap. MG | 650 | 725 | 894 | 197 | 325 |
| Res. Area, Acres | 105 | 109 | 102 | 26 | 69.5 |
| Spillway Data | | | | | |
| Elev., MSL | 431 ~~432~~ | 223 ~~224~~ | 398 | 223 ~~220~~ | 161 ~~157.5~~ |
| Freeboard, Ft. | 4.25 | 5.0 | 12.0 | 4.0 | 6.0 |
| Length, Ft. | 80 | 70 | 50 | 40 | 80 |

*Does not include drainage area above upstream dam.

Additional data are as follows:

<u>Bethany</u> - Gravity masonry section built in 1892, faced with concrete in 1931. Downstream embankment. Spillway on dam

crossed by bridge of limited headroom. Downstream channel not limiting.

Watrous - Lies two miles downstream from Bethany Dam on West River. Watrous is a gravity concrete section with an earth embankment on the downstream side. Its spillway is not obstructed and the channel leading from the spillway is not limiting. Watrous Dam is about 0.6 miles upstream from Lake Dawson.

Chamberlain - Chamberlain was built of earth on the Sargent River branch of the West River, with a masonry core wall, in 1891. It was raised 35 feet and a new spillway was constructed in 1958-59. It has a side channel spillway with ample downstream channel capacity.

Glen - Glen Dam is a gravity concrete structure on the Sargent River one and one-half miles below Chamberlain Dam.

Dawson - Dawson Dam was built in 1889. It is an earth structure with a concrete core wall. The spillway channel was damaged in the 1955 hurricane flood and rebuilt shortly thereafter.

**The West River continues to flow in a southerly direction below Lake Dawson, passing through Konolds Pond and between New Haven and West Haven to Long Island Sound, about six miles away.**

## III. REPORT OF STATE WATER RESOURCES COMMISSION

Mr. Joseph Cone's report considered flood experiences at each of the West River dams and estimated the flows that spillways of these dams could carry safely. The report did not include a detailed study and was in effect a reconnaissance study of the structures in question. A detailed study was left up to the Company, and this present report concerns more detailed studies of each dam and spillway.

Mr. Cone's conclusions are summarized as follows:

(1) A storm with a recurrence interval of 1,000 years probably should be used in studying dam safety.

(2) The most severe storm of record in the West River area, that of October 1955, was probably one with a recurrence interval of less than 100 years.

(3) The West River drainage area is approximately at the lower size limit of the Connecticut Formula. Flood flow from its smaller parts can probably be better estimated using the formula below:

$$Q = RF \times LF \times FF \times 9A^{2/3}$$

Q = Flow, cfs
RF = Rainfall Factor
LF = Ground Cover Factor
FF = Frequency Factor
A = Area in Acres

(4) Spillway capacities of the five reservoirs of the West River system are estimated as follows:

| Dam | cfs | csm |
|---|---|---|
| Bethany | 1,980 | 540 |
| Watrous | 2,660 | 380 |
| Chamberlain | 6,300 | 1,525 |
| Glen | 1,120 | 195 |
| Dawson | 2,870 | 215 |

(5) The report concludes as follows:

(a) Bethany should be able to carry a flow of over 4,000 cfs and with a 1,000-year storm would be overtopped by one foot.

(b) Watrous spillway will barely carry flood from its direct watershed and hence is deficient in capacity by the flow from Bethany or 4,000 cfs.

(c) Chamberlain has an adequate spillway.

(d) Glen was nearly overtopped in 1955 and will be overtopped by a greater storm.

(e) Dawson was nearly overtopped in 1955 and can be expected to be overtopped with any greater storm.

(6) It recommends a comprehensive study with corrective measures to be applied as soon as possible.

These estimates indicate that at peak flow the Bethany Reservoir is about 1.3 feet below the top of the dam; Chamberlain Reservoir is about 7.7 feet below the top of the dam; and Watrous Reservoir is about 0.3 feet below. Both Watrous and Bethany are masonry sections and little or no freeboard is essential, although some is usually allowed to prevent waves from splashing over the dam.

The spillway at Glen Dam will presently carry about 1,200 cfs before the dam is overtopped. It is estimated that this storm is of the magnitude that has a recurrence interval of about 300 years. The 1,000-year storm, as used in this report, would produce a reservoir elevation about 1.0 foot above the top of the dam. The dam is of masonry and could withstand overtopping. The overflow would be voluminous and would result in considerable erosion below the dam. In our opinion the risk is too great to continue operation of this reservoir with the present spillway capacity even though overtopping of this reservoir is not likely to cause danger to life and the property of others below the West River system. Methods of increasing spillway capacity are discussed in Section VI.

Dawson spillway will carry a flood of 3,620 cfs with no freeboard. With 2 feet of freeboard, the minimum we consider feasible for this dam, the spillway will carry about 1,900 cfs. The estimated outflow for the 1,000-year storm is 5,300 cfs. In our opinion the Dawson spillway can safely carry a storm with a frequency of about 150 years. Dawson is the lowest dam in the series on the West River system and is located above a populated and developed area that probably would suffer severe damage and possible danger to life in case of failure. As it is an earth dam that must not be overtopped, even by wave runup, its spillway capacity must be increased materially. Methods of doing so are discussed in Section VI.

Westfield, Massachusetts, Storm of 1955

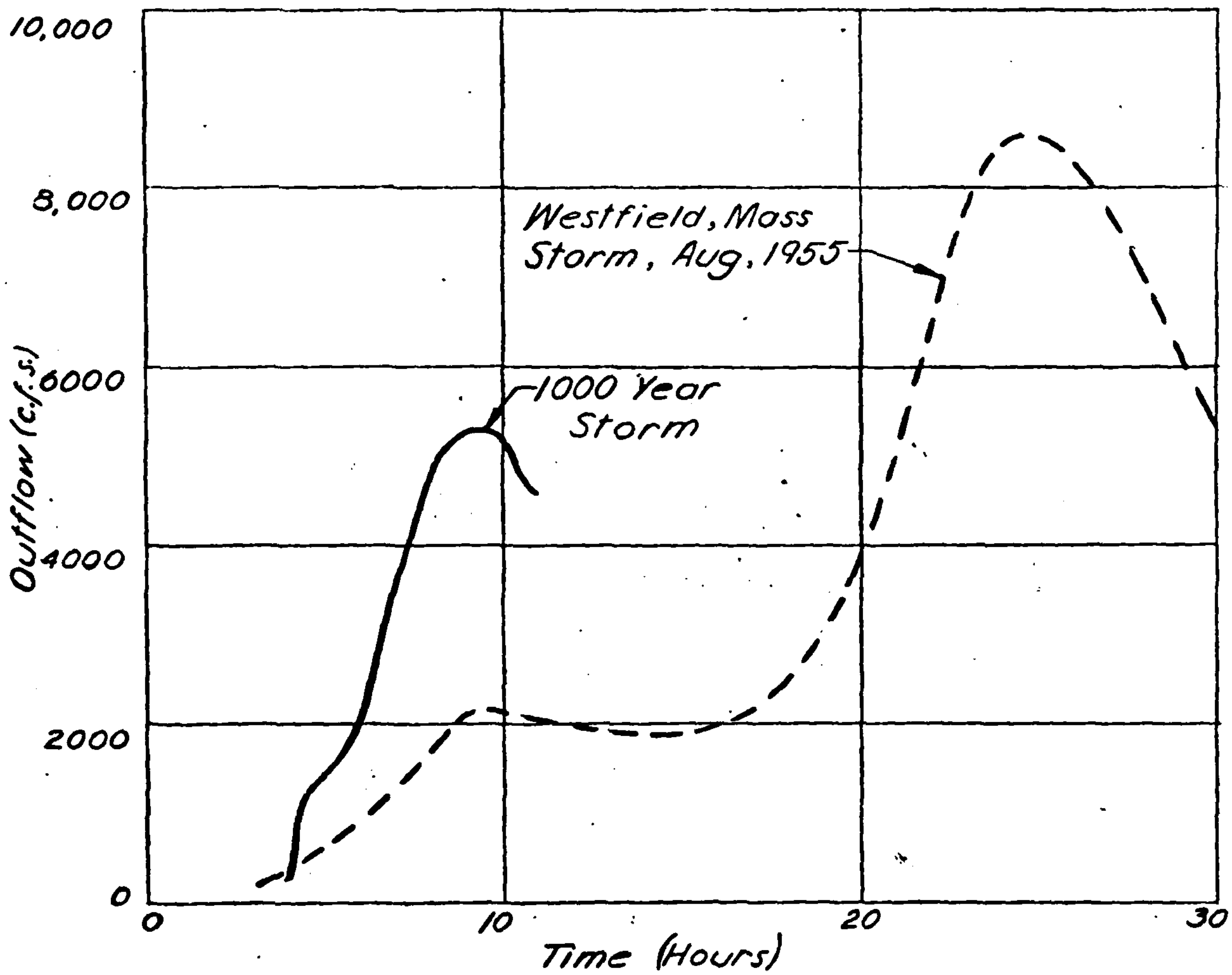
To investigate the effect of a storm similar to the Westfield, Massachusetts, storm of August 1955, the Norfolk, Connecticut, recording rain gage record of the storm was adjusted to equal the 24-hour readings taken at the Westfield gage and the resulting storm was transposed to the West River watershed. Hydrographs were constructed for runoff from the storm, which flows were routed through the reservoir system.

This storm produced much more water than the 1,000-year storm, and the peak flows are of the magnitude of 50 per cent greater. The following tabulation compares the two storms.

| Reservoir | Outflow from Reservoir, cfs 1,000-Year Storm | Westfield Storm | |
|---|---|---|---|
| Bethany | 1,500 | 2,200 | 1980 |
| Chamberlain | 1,800 | 2,700 | 6300 |
| Watrous | 2,800 | 4,300 | 2660 |
| Glen | 2,300 | 3,800 | 1200 |
| Dawson | 5,300 | 1'FB 8,700 | 3620 |

Figure 2 shows a visual comparison of the two storms in terms of outflow from Dawson Reservoir.

Fig. 2




DAWSON

COMPARISON OF OUTFLOW HYDROGRAPHS - 1000 YEAR STORM VS. WESTFIELD, MASS. 1955 STORM

The Westfield storm produces outflows within the spillway capacities of Bethany Wren and Chamberlain. Watrous is overtopped by about 0.2 feet. In view of the uncertainty of the estimates and the construction of the dam, this slight overtopping does not appear of great concern.

Both Glen and Dawson would be overtopped to a greater extent than in the 1,000-year storm, and this factor has been kept in mind in considering methods of increasing spillway capacity discussed in Section VI.

## VI. METHODS OF INCREASING SPILLWAY CAPACITY

In our opinion there is no need to consider modification of the Bethany, Chamberlain or Watrous spillways to provide additional capacity to carry flood flows. Serious consideration must be given to the effect of probable future flood flows at Glen and Dawson spillways.

### Glen Dam

For Glen spillway to carry the 1,000-year flood without overtopping the present dam and without use of the blowoff will require increasing the spillway length to 78 feet or, with present length, increasing the freeboard to 6.0 feet. To carry the Westfield storm requires increasing the spillway length to 95 ~~feet or~~ the freeboard to 9.2 feet using the present length. The factor of safety against overturning for Glen Dam, as determined in Section VIII, is as small as can be tolerated when the water level is 4 feet above the spillway crest, so raising the dam does not appear feasible.

Spillway a.
(Elev. 218.8
(spillway
215.63

It appears possible to add the required length by building an extension to the existing spillway at a 90 degree angle or by installing an auxiliary spillway at the north end of the dam. Either is feasible, although there are advantages to confining such work to the present spillway location so a common discharge channel may be used. The existing spillway may also be replaced by a side channel spillway 95 feet long.

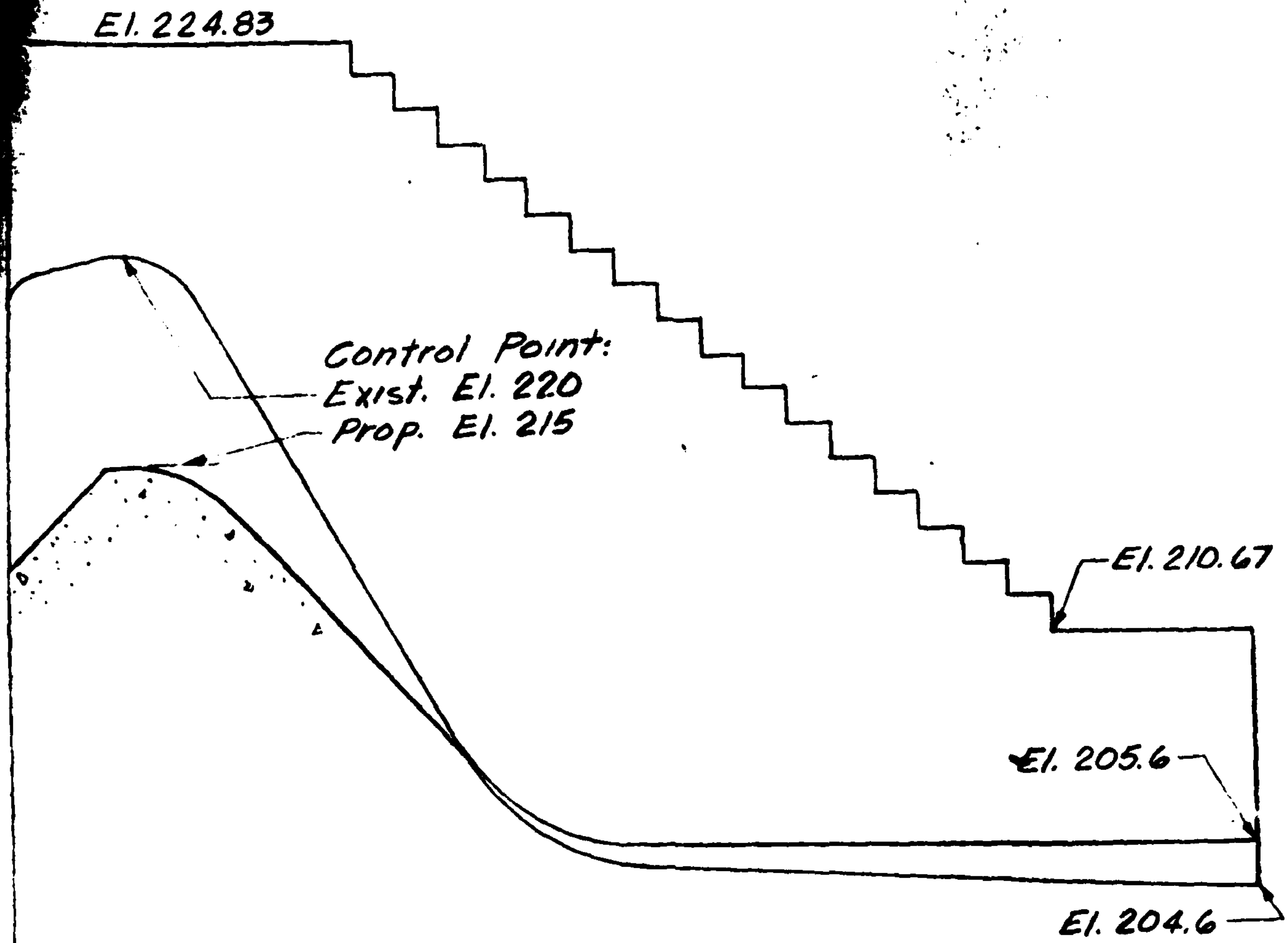
Glen Reservoir has a small storage capacity and is principally used to provide workable head conditions for the Sargent River portion of the West River system. Lowering the normal reservoir level 5 feet would decrease storage by about 40 million gallons and would reduce the yield of the West River system a very small amount. At full reservoir, pressures would be reduced about 2 psi.

Although the urgency of providing more outflow capacity for Glen is not as great as for Dawson, it is advisable to modify the Glen spillway at an early date. The least expensive method of doing so appears to be by lowering the spillway crest as shown in Figure 3 to permit passing future extreme floods without overtopping the non-overflow section of the dam. For the 1,000-year storm used in our study, the spillway should be lowered at least 2 feet. For the Westfield storm it should be lowered 5 feet. In view of the uncertainty of all methods of estimating future flood conditions and the minor effect on system operation if this plan is followed, we recommend lowering the Glen spillway by 5 feet. The cost of cutting down and reshaping the spillway crest is estimated to be of the magnitude of $5,000. The work does not require extensive preparation and can be started at any time.

Crest gates could be installed on the spillway after lowering to maintain present storage. They would add approximately $100,000 to the cost.

An alternate method of obtaining the necessary spillway capacity while maintaining present water levels would be to rebuild the spillway. This alternate will cost about $100,000, approximately the same as the crest gate alternate. Given the

Fig. 3




SECTION THROUGH SPILLWAY
SCALE: 1/4" = 1'-0"

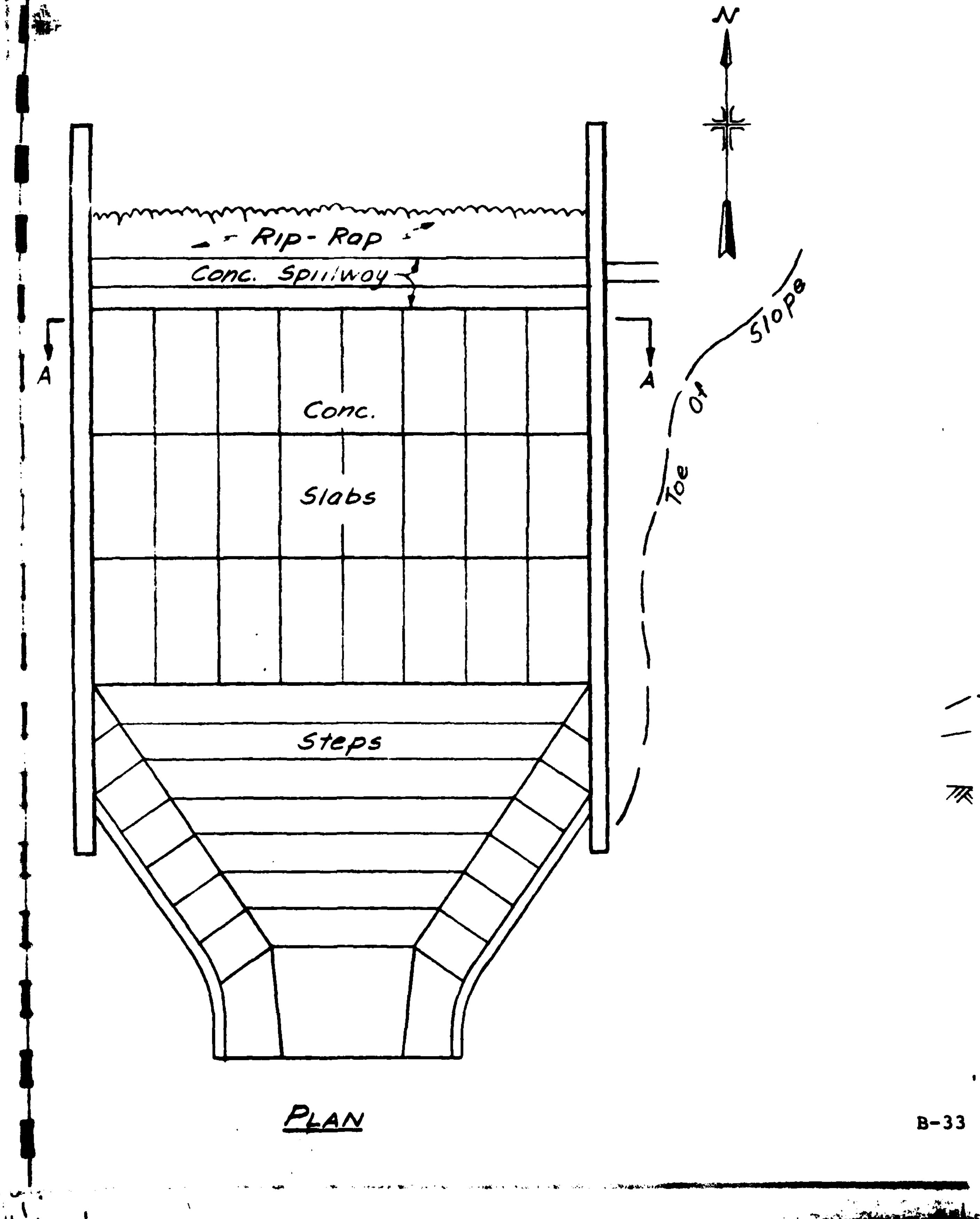
PROPOSED METHOD OF
LOWERING GLEN SPILLWAY

two choices, we prefer extending the fixed spillway rather than utilizing crest gates with their attendant maintenance and operation problems.

Dawson Dam

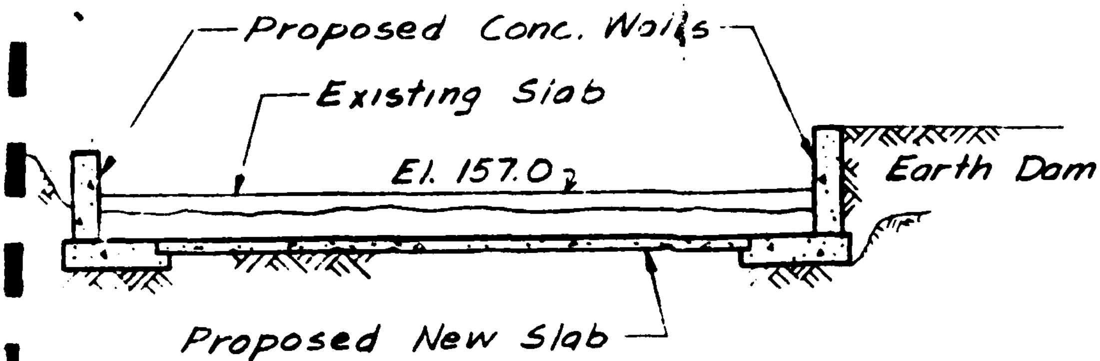
The spillway at Dawson Dam now is 80 feet long. To carry a flood of 5,300 cfs, without freeboard, the spillway must be lengthened by about 40 feet. With 2 feet of freeboard, the length must be increased 140 feet. To carry the Westfield storm of 8,700 cfs without freeboard would require extending the spillway 115 feet. Extensions beyond about 30 feet by projection of the spillway line are difficult because of topographical conditions. Extending a side channel spillway northward alongside of the reservoir would necessitate channel construction through the existing spillway channel. Detailed studies have not been made, but preliminary examination indicates that it will be less costly to lower the existing spillway. If the spillway is lowered 5 feet and 2 feet of freeboard are allowed, it will carry a flow of about 7,200 cfs. This is more than the 1,000-year flood of 5,300 cfs and less than the Westfield storm of 8,700. The Westfield storm would reduce freeboard to about 1 foot. The cost of lowering the spillway 5 feet would be somewhat greater than lowering it 3 feet, which would allow no freeboard for the 1,000-year storm, but the major difference would be in rock excavation, and the added safety would be worth the difference in cost. We recommend lowering the spillway 5 feet as shown in Figure 4, at an estimated cost of $125,000.

Fig 4

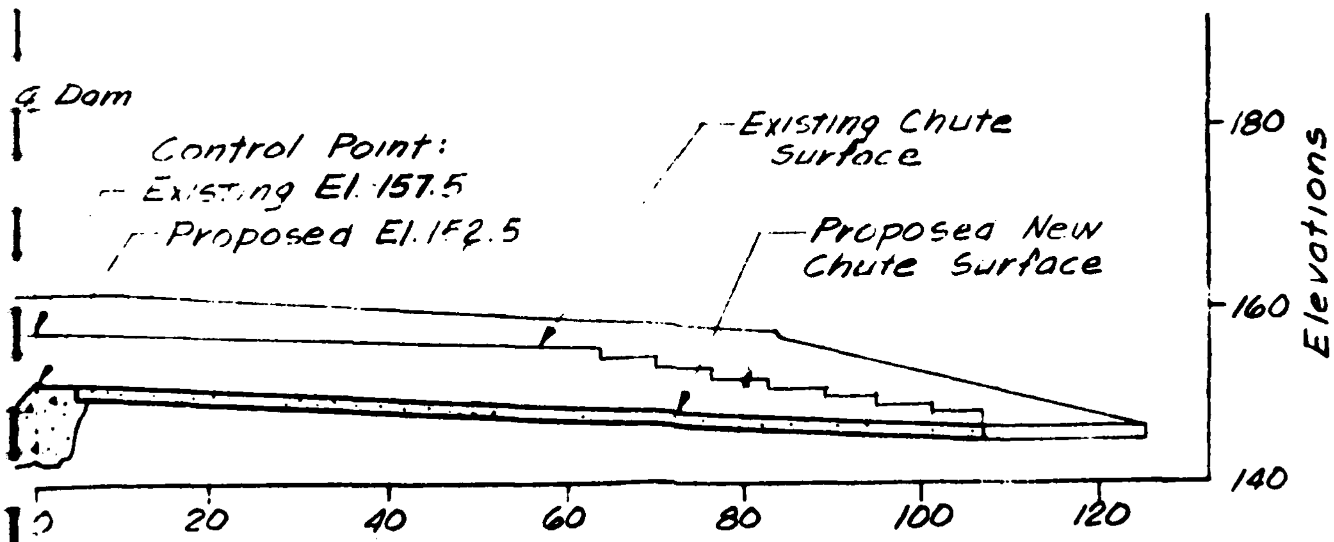



PLAN

Fig. 4




SECTION A A




LONG. SECTION

PROPOSED METHOD OF
LOWERING DAWSON SPILLWAY
SCALE: 1" = 20' HORIZ & VERT.

Dawson Reservoir is at too low an elevation for direct service, and its yield is now pumped into the system when needed. Lowering the spillway 5 feet would reduce storage by about 110 million gallons and would reduce slightly the yield of the West River system. Other considerations may indicate the need of maintaining water levels at present flow line elevation. If so, crest gates may be installed at a cost of approximately $150,000, making the total cost of the work approximately $275,000.

## VII. EFFECT ON YIELD OF LOWERING SPILLWAY CRESTS

If Glen spillway is lowered 5 feet, storage is reduced about 40 mg. If Dawson is lowered 5 feet, the loss in storage is 102 mg. Total storage loss if both spillways are lowered is 142 mg.

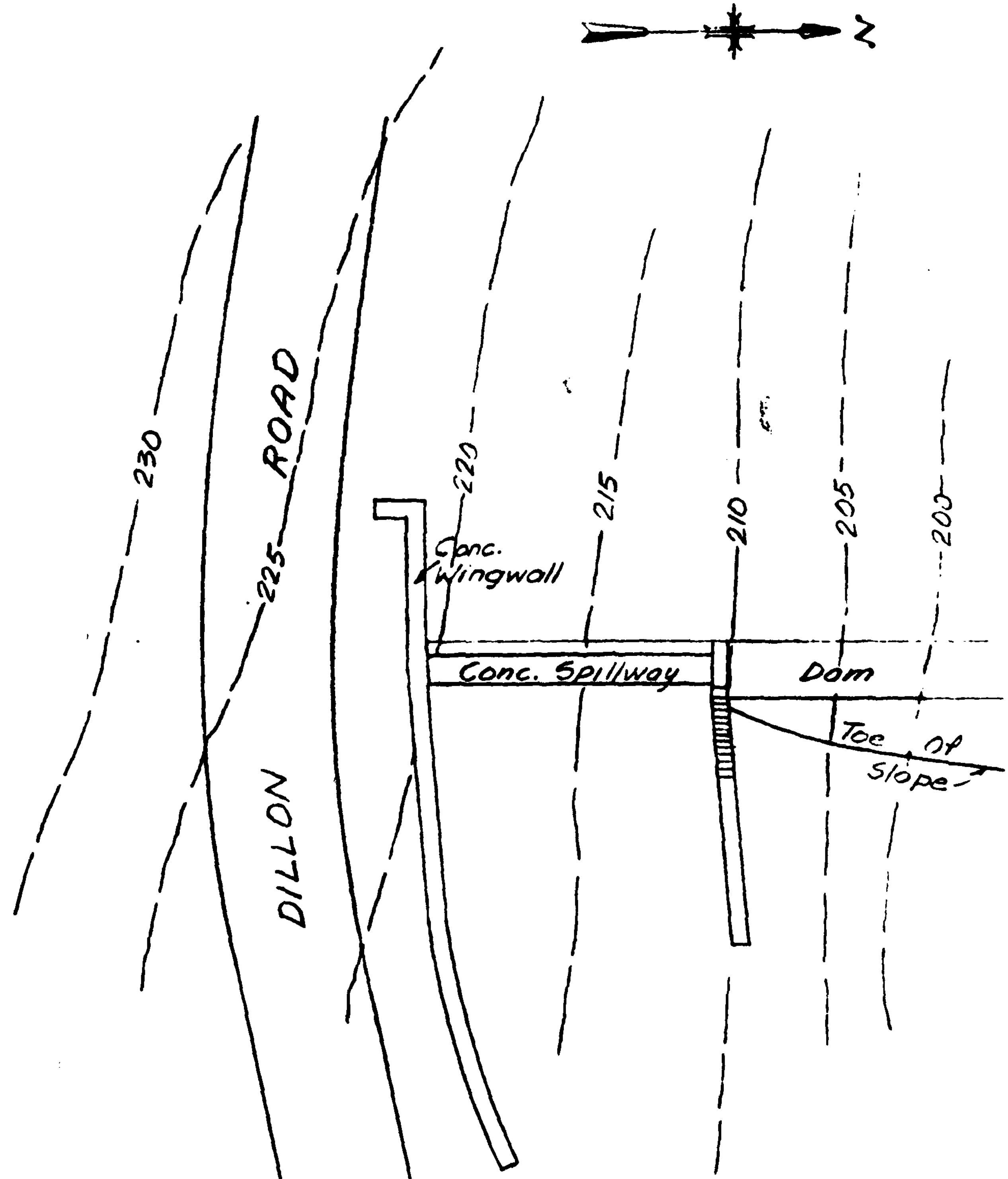
During the 1964-66 dry period, water produced from May 20, 1964, to November 1, 1966, averaged about 8.1 mgd. The minimum amount left in storage in February 1966 was about 626 mg. With no reserve allowance, and assuming that the reservoirs refill by next June, the supply could have been increased about 0.6 mgd and the system yield would be 8.7 mgd. If 20 per cent storage was allowed for emergency reserve, the yield would be approximately 8.2 mgd.

The loss in storage by lowering the spillway would have decreased yield over this dry period by 0.13 mgd. During wet periods when the system refills each year, loss of yield would be greater and, in a year when Dawson is below flow line level for a 6-month period, the reduction would be about 0.8 mgd.

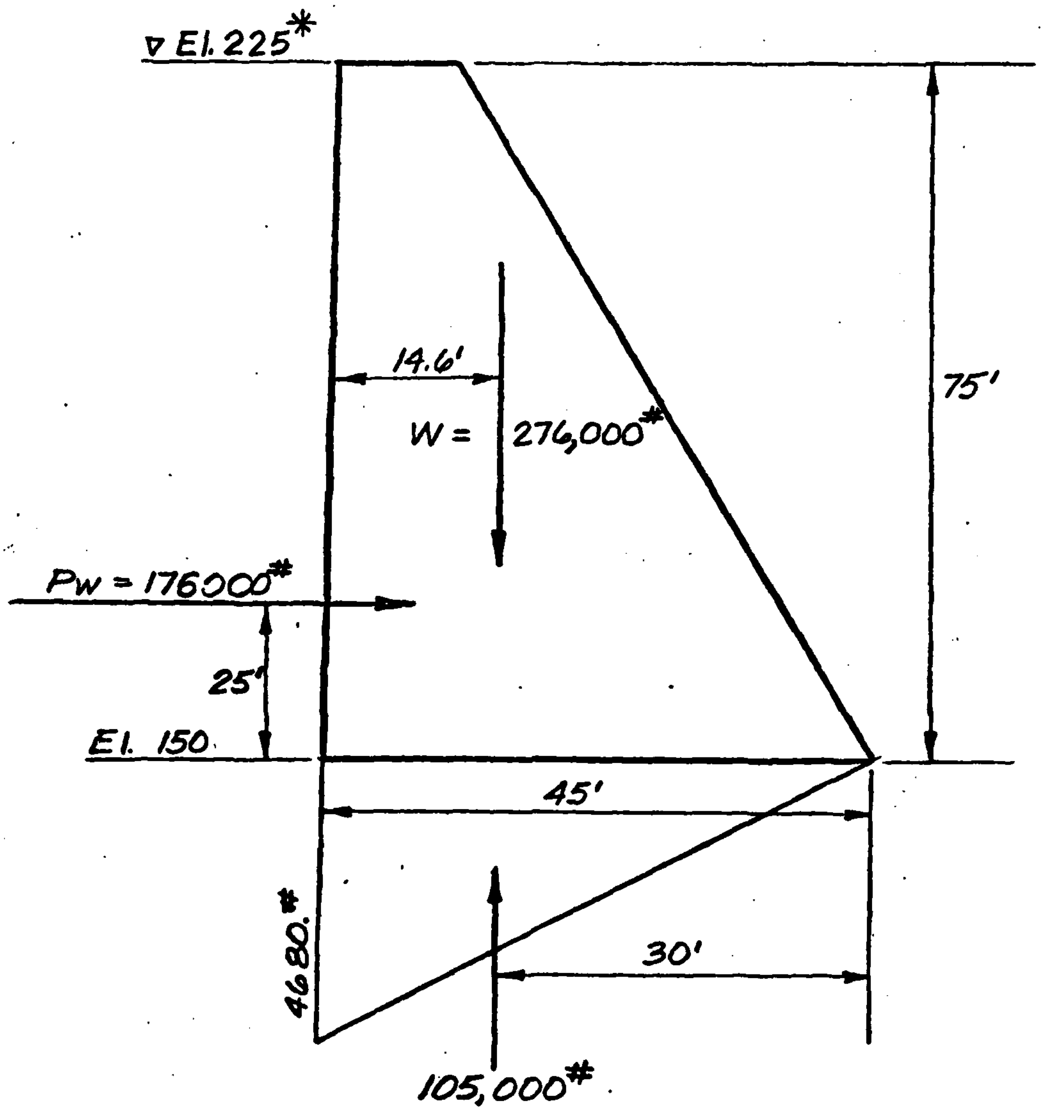
# VIII. STABILITY OF DAMS

Stability of each dam in the West River system has been investigated. Chamberlain and Dawson are earth dams with satisfactory sections. Bethany and Watrous are masonry dams with massive earth backup on the downstream side. There is no question as to their stability.

Glen is an exposed masonry dam and its stability has been investigated against overturning. Because of the construction, it is safe against sliding. When full to the crest of the non-overflow section, presently 4 feet above the spillway elevation, the factor of safety against overturning is 1.18. If the dam is raised one (1) foot, which could be easily done, the factor of safety decreases to 1.11, as shown in Figure 5. Since uplift is probably less than assumed, we estimate that the dam is safe against overturning as long as the maximum water level does not exceed the top elevation of the existing non-overflow section. We do not recommend any increase in height.




PLAN
SCALE: 1" = 20'




F.S. - OVERTURNING = $\frac{276 \times 30.4}{(176 \times 25) + (105 \times 30)} = 1.11$
(With top El. 225 - assuming dam raised one(1) foot.)

STABILITY ANALYSIS – GLEN DAM

*Present Elevation 224

## IX. RECOMMENDATIONS

We recommend that the Water Company increase the capacity of the spillways at Glen and Dawson Dams by lowering each of these spillways approximately five (5) feet. Since a major storm may occur at any time, the work should be done as soon as possible.

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT



MEMORANDUM REPORT TO WATER COMPANY
ON
INVESTIGATION OF THE EFFECTS OF A FLOOD
PRODUCED BY THE MAXIMUM POSSIBLE STORM
ON SPILLWAYS OF WEST RIVER SYSTEM

AUGUST 2, 1967

The effect of the "maximum possible storm" on the West River System is reported in this memorandum.

The "maximum possible storm" employed is defined and quantitatively estimated in U. S. Weather Bureau Hydrometeorological Report No. 33 entitled "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours." The report defines the "maximum possible precipitation" as "the critical depth-duration-area rainfall relation for a particular area during various months of the year that would result if conditions during an actual storm in the region were increased to represent the most critical meteorological conditions that are considered probable of occurrence."

As shown on Exhibit 1, the rainfall totals used for the West River System analyses are for durations of 6 and 12 hours on an area of 10 square miles for September -- the most severe month for the vicinity of New Haven, Connecticut. The hourly distribution of the total rainfall assumed is according to Figure 4, page 32 of U. S. Department of the Interior publication "Design of Small Dams." The distribution is a comparatively severe one with 50 per cent of the 6 hour total falling within 1 hour.

The sequence in which the hourly totals were arranged is in accordance with the recommendation made on page 50 in "Design of Small Dams." The arrangement of the 12 hourly increments is 11, 9, 7, 5, 3, 1, 2, 4, 6, 8, 10, 12, where the number represents the order of magnitude with the lowest number representing the largest magnitude. This arrangement gives a flood greater than one based on the assumption that the greatest hourly increment of rain occurs during the first hour of a storm

The effective, runoff-producing rainfall was estimated by subtracting 1 inch initial infiltration and 0.1 inch per hour thereafter from the total rainfall.

In order to pass the unusually high flows for the "maximum possible storm," several modifications of both the length and crest height of spillways were tried. Spillway rating curves and stage capacity curves for each of the five reservoirs are shown on Exhibit 2 and Exhibit 3, respectively.

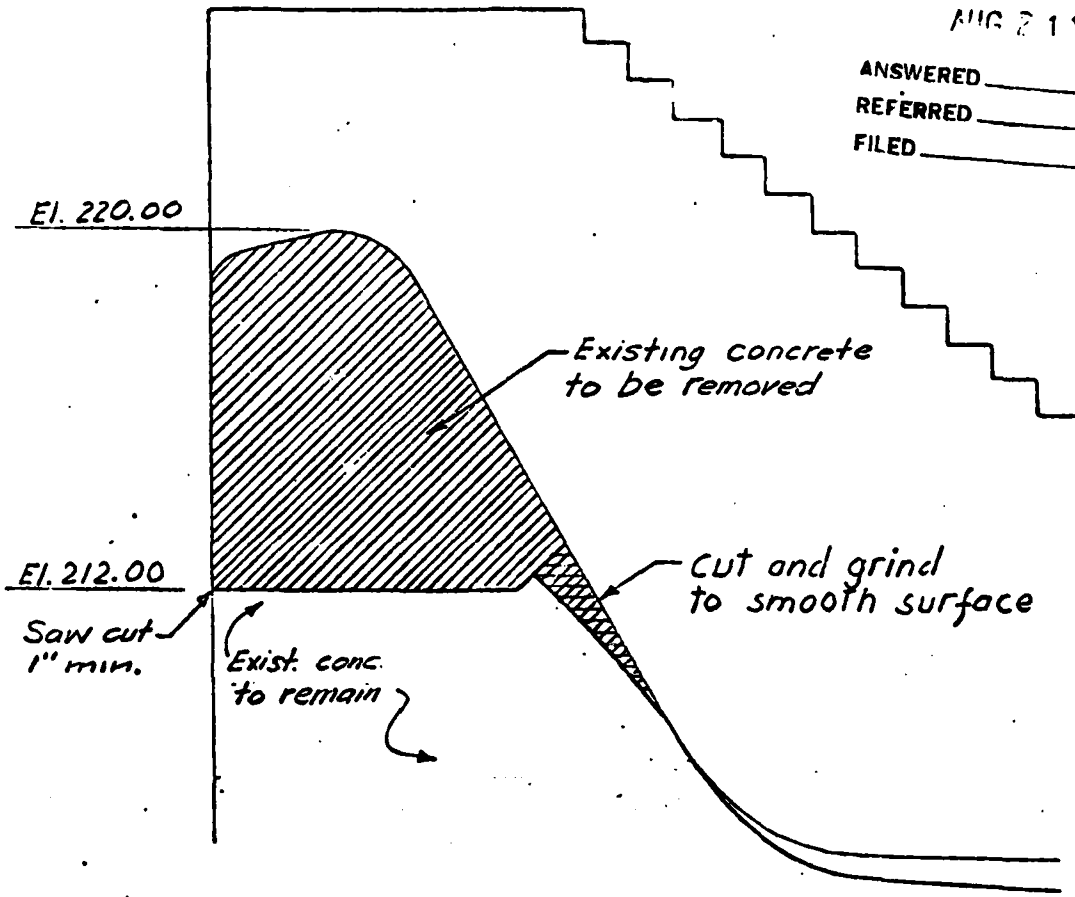
The unit-hydrographs and routing procedures employed are those outlined in our report of January, 1967. Detailed computations are shown on Exhibit 4, pages 1 through 8.

The inflow-outflow curves for each of the reservoirs are shown on Exhibit 5, pages 1 though 3. As no significant storage effect is obtained from Lake Dawson, the outflow hydrograph as shown on Exhibit 5, page 3, will be the same with a spillway 250 feet long.

The "maximum possible" flood outflows at each of the West River reservoirs and the conditions at the Spillways are summarized below:

| Dam | Peak Spillway Discharge cfs | Free-Board ft. | Maximum Head (ft.) Over Spillway | Over Dam Crest |
|---|---|---|---|---|
| Chamberlain | 7200 | 12.0 | 10.8 | -1.2 |
| Glen | 9665 | 9.0* | 11.3 | +2.3 |
| Bethany | 7350 | 4.25 | 5.2 | +1.0 |
| Watrous | 15,400 | 5.0 | 7.1 | +2.1 |
| Dawson | | | | |
| 80' Spillway | 26,260 | 11.5* | 13.8 | +2.3 |
| 250' Spillway | 26,260 | 11.0* | 9.0 | -2.0 |

*Freeboard above proposed new sill elevation




EXISTING SECTION
1/4" = 1'-0"

LENGTH OF SPILLWAY 40'±

NEW HAVEN WATER CO.
NEW HAVEN CONN.

LOWERING OF
GLEN SPILLWAY

REMOVAL OF
EXISTING CREST

M.O'S

MALCOLM PIRNIE ENGINEERS

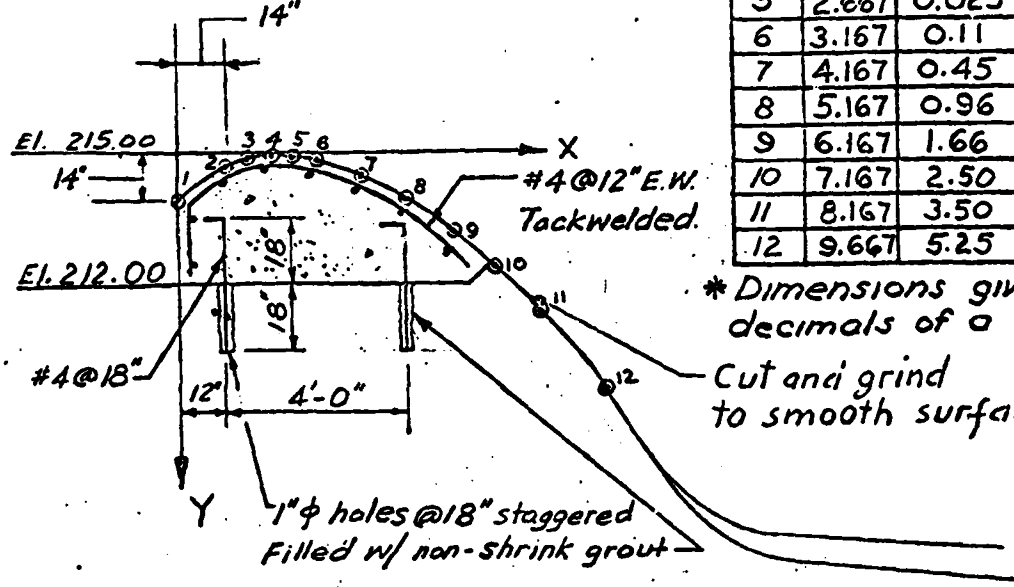
DEC. 1967

DWG. NO. 56C-67.001-0

TABLE OF SPILLWAY COORDINATES

| PT. | X | Y |
|---|---|---|
| 1 | 0 | 1.167 |
| 2 | 1.167 | 0.22 |
| 3 | 1.667 | 0.05 |
| 4 | 2.167 | 0.0 |
| 5 | 2.667 | 0.025 |
| 6 | 3.167 | 0.11 |
| 7 | 4.167 | 0.45 |
| 8 | 5.167 | 0.96 |
| 9 | 6.167 | 1.66 |
| 10 | 7.167 | 2.50 |
| 11 | 8.167 | 3.50 |
| 12 | 9.667 | 5.25 |

* Dimensions given in decimals of a foot




NEW SPILLWAY SECTION
1/4" = 1'-0"

LENGTH OF SPILLWAY 40'±

NEW HAVEN WATER CO.
NEW HAVEN, CONN.

LOWERING OF GLEN SPILLWAY

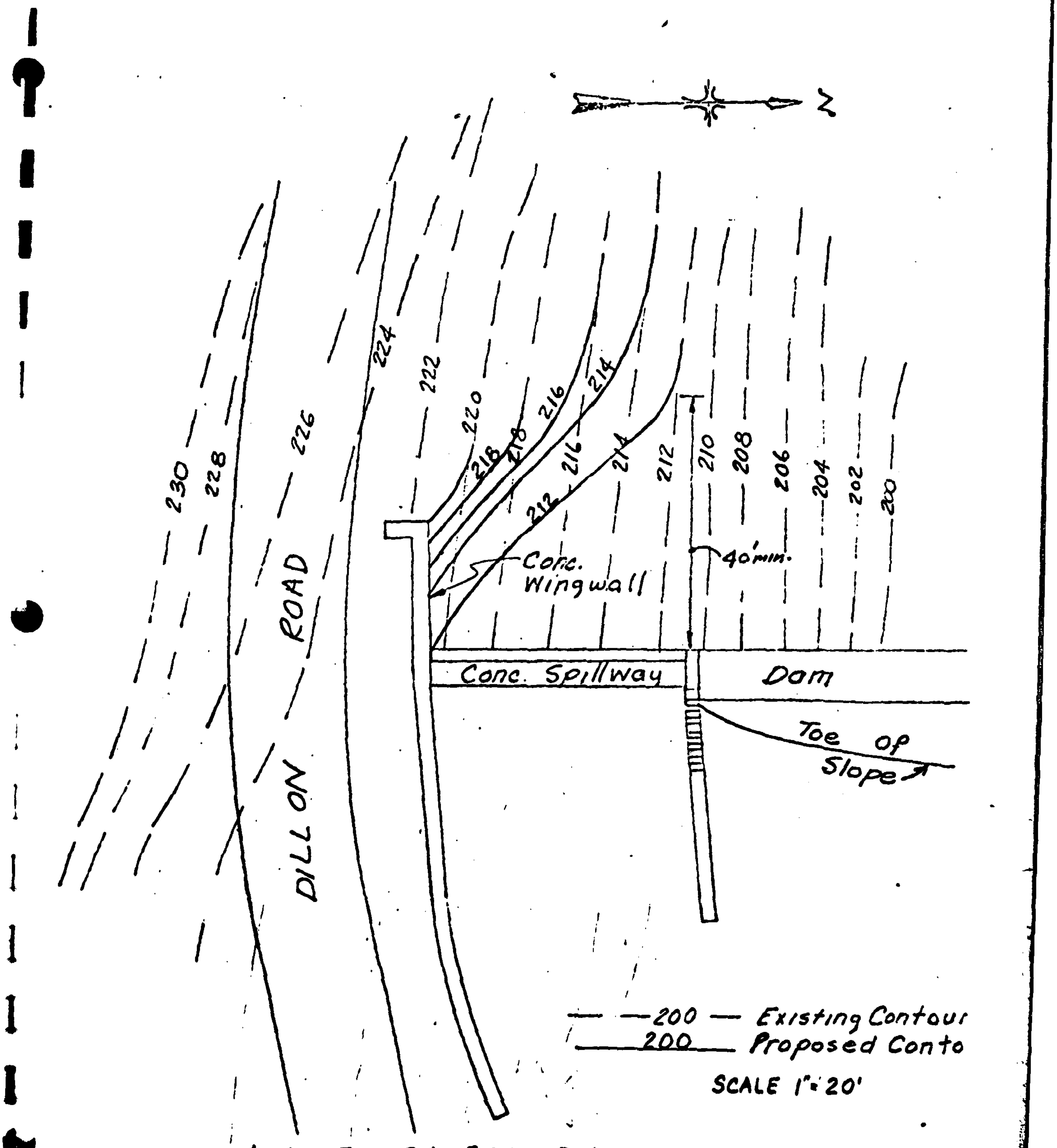
NEW PROFILE

M.O'S.

MALCOLM PIRNIE ENGINEERS

DEC. 1967

DWG. NO. 56C-67.002-0




# LAKE GLEN DAM

## SPILLWAY MODIFICATIONS
## GRADING PLAN

State Office Building
Hartford, Connecticut

APPLICATION FOR CONSTRUCTION PERMIT FOR DAM

...r New Haven Water Company — Date July 26, 1968

...dress Box 1470

New Haven, Connecticut — Tel. No. 203-772-2550

...tion of Structure:

Woodbridge (Glen Dam) — Shown on USGS Quadrangle New Haven and Vicinity, Conn.

...of Stream Sargent River — at 1/2 inches ~~xxxxx~~ north of Lat. 41°-22'-30"

and 2-1/2 inches east of Long. 73° 00' ~~xxxx~~

...ctions for reaching site from nearest village or route intersection:
(... sketch on reverse side)

See Attached Plans

... is an application for: (New Construction) (Alteration) ✓ (Repair) (Removal)
(check one or more of above)

... pond is to be used for: Storage

...nsions of Pond: width 330 ft. avg. length 3,000 ft. area 23 acres

... depth of water immediately above dam: 46 feet.

...ngth of dam: 370 ft.

... of spillway: 40 ft.

...ht of abutments above spillway: 9 ft.

... of spillway construction: Concrete

... dike construction: Gravity Concrete

...way section will be set on: (Bedrock) ✓ (Gravel) (Clay) (Till)
(check one of above)

...rks: Existing spillway section is set on bedrock

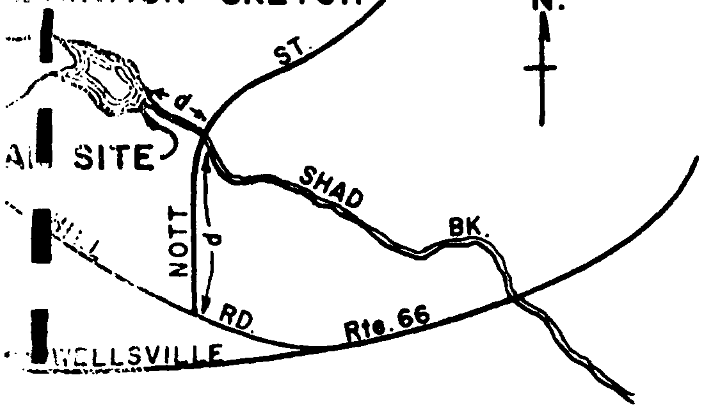
Signed: Joseph G. Novaro, Chief Engr.
(owner)

Name of Engineer, if any MALCOLM PIRNIE ENGINEERS

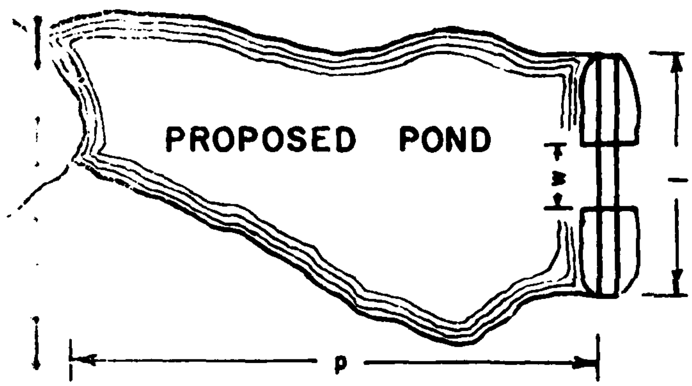
... Show details of
...ction on reverse side

## SAMPLE DATA

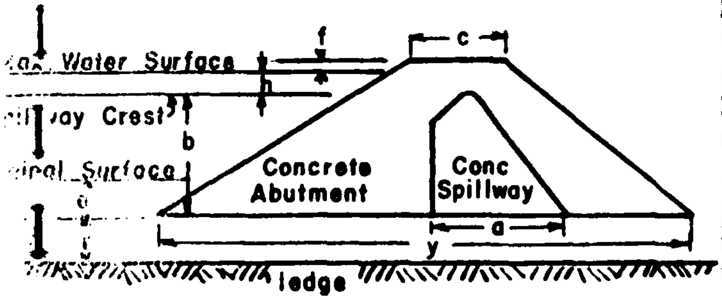
### LOCATION SKETCH



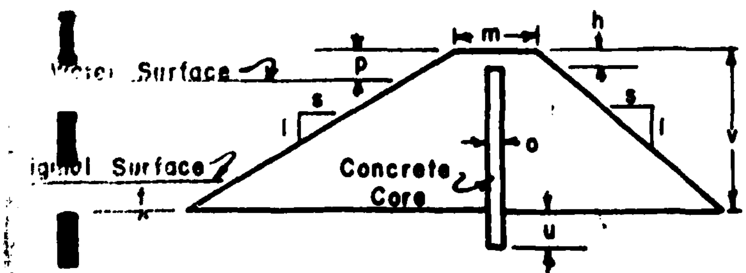

### SITE PLAN




### SPILLWAY SECTION




### DIKE SECTION




## APPLICANT'S DA

Show only features of sample which are applicable and dimensions which reflect your

### LOCATION SKETCH

See Location Plan on Sheet 1 of Lake Dawson Plans.

### SITE PLAN

l = 370'
w = 40'
p = 3,000'

### SPILLWAY SECTION

a = 13' min. (varies)
b = 60' max. (varies)
c = 9'
e = 2' min. (varies)
f = 3.0'
h = 6.0' (1,000 year storm)
r = 0
y = 45'

NOTE...
If there are two methods of discharge Show

### DIKE SECTION (Existing Gravity Concrete)

m = 9'
p = 9'
t = 20' max (varies)
v = 70' max (varies)

August 20, 1968

MEMO TO: File

FROM: William H. O'Brien III

SUBJECT: GLEN DAM - WOODBRIDGE

The following information was obtained from Mr. Raymond Dugandzic of Malcolm Pirnie Engineers in the process of the design review of the spillway modifications.

The proposed spillway will accomodate the outflow from the transposed Westfield storm with the water level at the top of the dam (no freeboard.)

W. H. O'Brien III

William H. O'Brien III

**NEW HAVEN WATER COMPANY**
**NEW HAVEN, CONNECTICUT 06506**

STATE WATER RESOURCES COMMISSION
RECEIVED
AUG 21 1969
ANSWERED ____
REFERRED ____
FILED ____

August 19, 1969

State Water Resources Commission
State Office Building,
Hartford, Conn.

Ref: Glen Dam on Sargent River
Woodbridge, Conn.
Construction Permit Aug. 20, 1968

Gentlemen:

This is to advise that the lowering of the spillway of Glen Dam has been completed in accordance with the original drawings furnished to your office, copies enclosed.

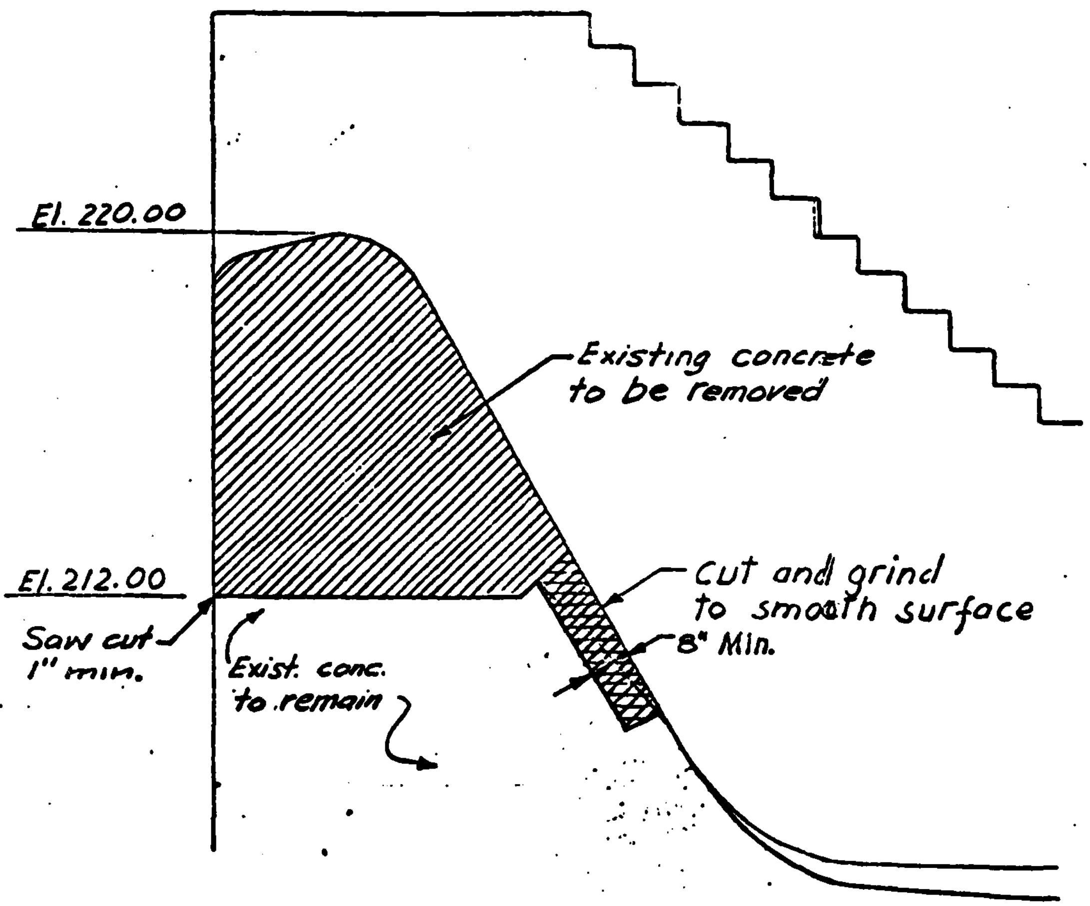
In addition, regrading the upstream side of the spillway was accomplished in accordance with enclosed Grading Plan in order to provide adequate approach channel to the lowered spillway. Rock removal was accomplished by line drilling, wedging and barring. Explosives were not used.

Our Mr. Donald Jackson and I will be available for your final inspection at your convenience.

Yours very truly,
NEW HAVEN WATER COMPANY

Joseph A. Novaro
Joseph A. Novaro
Chief Engineer

Copy to Mr. D. Jackson




EXISTING SECTION
1/4" = 1'-0"

LENGTH OF SPILLWAY 40'±

NEW HAVEN WATER CO.
NEW HAVEN CONN.

LOWERING OF
GLEN SPILLWAY

REMOVAL OF
EXISTING CREST

RECORD DWG.
R.J.D. 6/15/73.

M.O'S

MALCOLM PIRNIE ENGINEERS

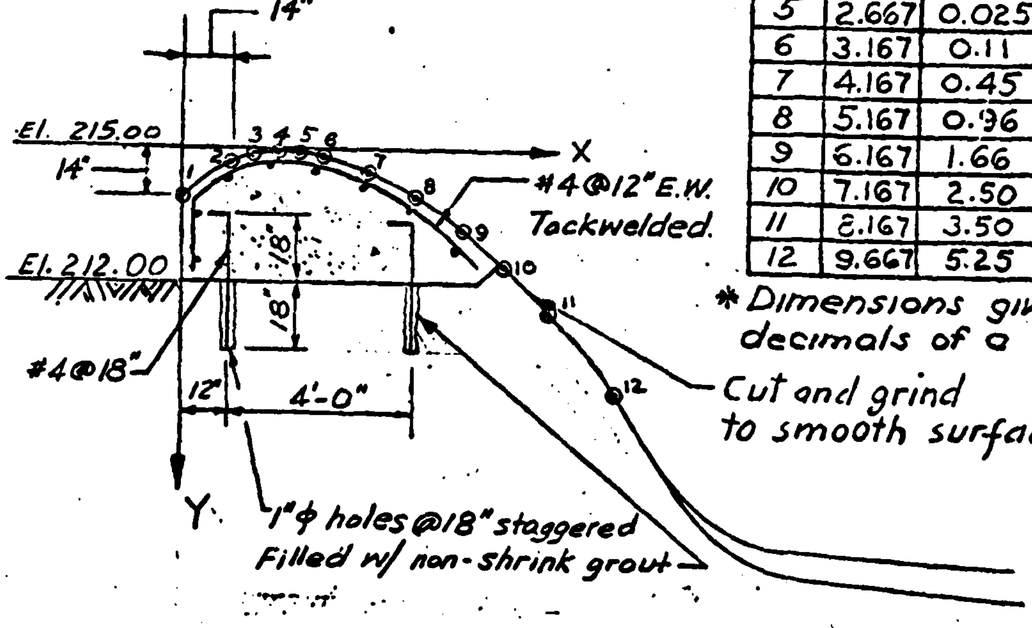
DEC. 1967

DWG. NO. 56C-67.001-1

## TABLE OF SPILLWAY COORDINATES

| PT. | X | Y |
|---|---|---|
| 1 | 0 | 1.167 |
| 2 | 1.167 | 0.22 |
| 3 | 1.667 | 0.05 |
| 4 | 2.167 | 0.0 |
| 5 | 2.667 | 0.025 |
| 6 | 3.167 | 0.11 |
| 7 | 4.167 | 0.45 |
| 8 | 5.167 | 0.96 |
| 9 | 6.167 | 1.66 |
| 10 | 7.167 | 2.50 |
| 11 | 8.167 | 3.50 |
| 12 | 9.667 | 5.25 |

* Dimensions given in decimals of a foot




NEW SPILLWAY SECTION
1/4" = 1'-0"

LENGTH OF SPILLWAY 40'±

RECORD DWG.
R.J.D. 6/5/73.

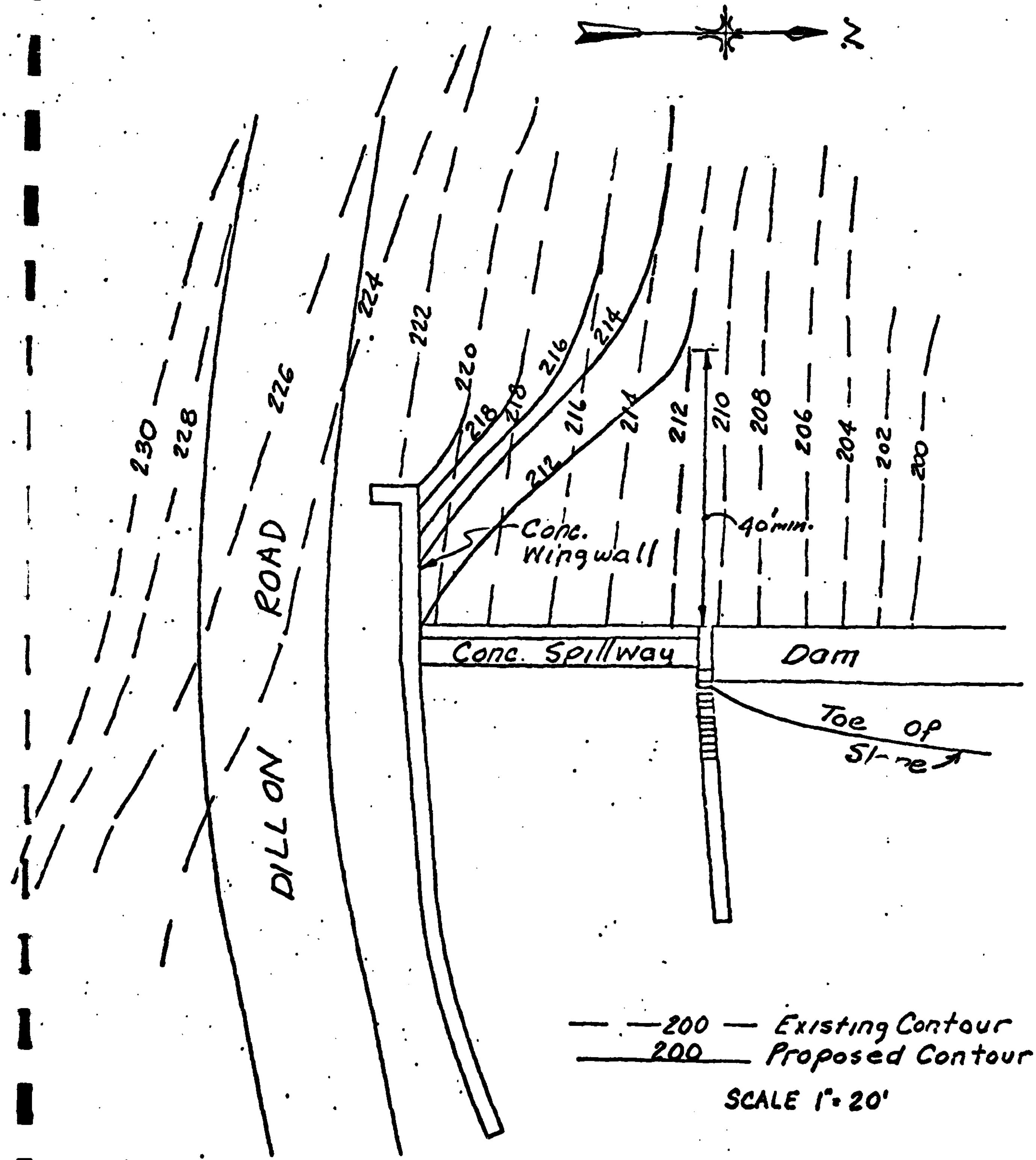
M.O'S.

MALCOLM PIRNIE ENGINEERS

NEW HAVEN WATER CO.
NEW HAVEN, CONN.

LOWERING OF
GLEN SPILLWAY
NEW PROFILE

DEC. 1967 DWG. NO. 56C-67.002-1




# LAKE GLEN DAM

## SPILLWAY MODIFICATIONS
## GRADING PLAN

RECORD DWG.
R.J.D. 6/5/73

DEPARTMENT MESSAGE

200 2/69

SAVE TIME: *Handwritten messages are acceptable.*
*Use carbon if you really need a copy*

| | AGENCY | DATE |
|---|---|---|
| File | Water Resources Commission | March 1, 1971 |
| William H. O'Brien, III<br>Civil Engineer | AGENCY<br>Water Resources Commission | TELEPHONE |

JECT Glen Lake Dam, Woodbridge

On Feb. 24, 1971 the undersigned observed at least two evergreens, approximately 8 to 12 inches in diameter growing within approximately 3 feet of the downstream concrete wall of this structure. The dam is approximately 25 feet in height at this point which effectively shields the bottom half of the trees from wind, however, there is some 30 to 40 feet of the top of the tree exposed to wind and it is assumed that the roots could conceivably cause a deteriotion of the concrete wall.

W.H. O'Brien
Civil Engineer

WHO:ljg

Take closer look when inspecting for certificate of approval — W.H.O. // Inspected 6/19/73 by J.O.E. not considered ~~[illegible]~~ serious as dam is ledge, but [illegible].

SAVE TIME: *If convenient, handwrite reply to sender on this same sheet.*

29 June 1973

Mr. Richard P. McHugh
New Haven Water Company
100 Crown Street
New Haven, CT 06506

Re: Dawson Lake Dam
Glen Dam
Woodbridge

Dear Mr. McHugh:

Enclosed please find the Certificates of Approval for the subject dams.

During our inspection we found that the construction work was performed according to the plans submitted.

Although we are not ordering their removal, from the standpoint of good maintenance practices, the trees at the base of the Glen Dam should be removed.

Very truly yours,

Victor F. Galgowski
Supt. of Dam Maintenance
Water & Related Resources

VFG:ljg

Enclosures

STATE OF CONNECTICUT
DEPARTMENT OF ENVIRONMENTAL PROTECTION
STATE OFFICE BUILDING HARTFORD, CONNECTICUT 06115
WATER AND RELATED RESOURCES

29 June 1973

CERTIFICATE OF APPROVAL

DAN W. LUFKIN
COMMISSIONER

New Haven Water Company
100 Crown Street
New Haven, CT 06506

Attention: Mr. Richard P. McHugh

TOWN: Woodbridge
RIVER West River
TRIBUTARY: Sargent River
CODE NO.: W-1

Dear Mr. McHugh:

NAME AND LOCATION OF STRUCTURE: This dam is known as Glen Dam and is located on the Sargent River in the town of Woodbridge.

DESCRIPTION OF STRUCTURE AND WORK PERFORMED: Alterations consisted of the removal of the top of the existing spillway and replacing with a new crest. The length of the spillway is approximately forty feet.

CONSTRUCTION PERMIT ISSUED UNDER DATE OF: August 20, 1968

This certifies that the work and construction included in the plans submitted, for the structure described above, has been completed to the satisfaction of this department and that this structure is hereby approved in accordance with Section 25-114 of the 1971 Supplement to the General Statutes.

The owner is required by law to record this Certificate in the land records of the town or towns in which the structure is located.

Dan Lufkin

Dan W. Lufkin
Commissioner

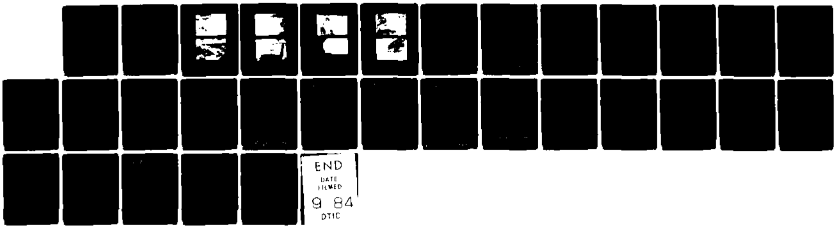
APPENDIX C

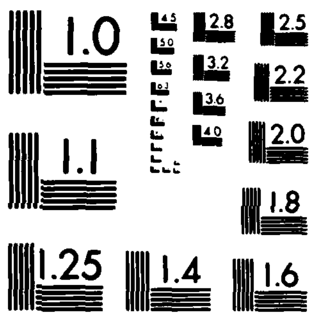
DETAIL PHOTOGRAPHS

AD-A144 190 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS GLEN LAKE DAM (CT 003..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV AUG 79

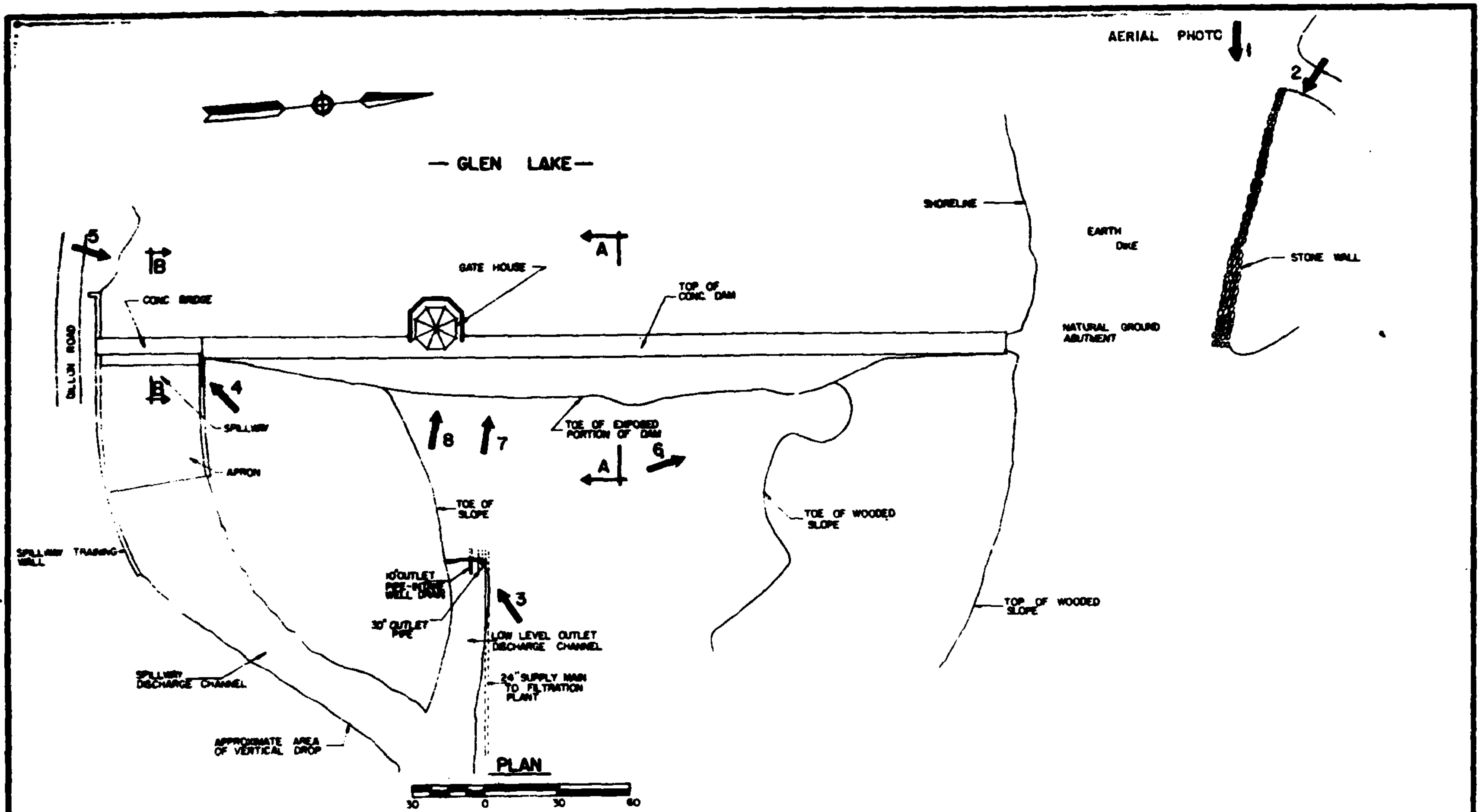
22

UNCLASSIFIED F/G 13/13 NL






MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A




# PHOTO LOCATION PLAN

## GLEN LAKE DAM

SHEET C-1

PHOTO 1 - Upstream view of dam. Note earth dike adjacent to left abutment.

PHOTO 2 - Left dike from upstream end. Note rubble stone wall on downstream side of dike.

| | | |
|---|---|---|
| US ARMY ENGINEER DIV. NEW ENGLAND<br>CORPS OF ENGINEERS<br>WALTHAM, MASS | NATIONAL PROGRAM OF<br>INSPECTION OF<br>NON-FED. DAMS | Glen Lake Dam<br>Sargent River<br>Woodbridge, Connecticut |
| CAHN ENGINEERS INC.<br>WALLINGFORD, CONN.<br>ENGINEER | | CE# 27 660 KA<br>DATE May '79 PAGE C-1 |

PHOTO 3 - Stone headwall with 30 inch low level outlet pipe and 8 inch well drain outlet pipe at downstream toe of dam below gatehouse.

PHOTO 4 - Concrete spillway weir spanned by foot bridge. Note cracking and effloresence of abutment training wall.

| | | |
|---|---|---|
| US ARMY ENGINEER DIV. NEW ENGLAND<br>CORPS OF ENGINEERS<br>WALTHAM, MASS. | NATIONAL PROGRAM OF<br>INSPECTION OF<br>NON-FED. DAMS | Glen Lake Dam<br>Sargent River<br>Woodbridge, Connecticut |
| CAHN ENGINEERS INC.<br>WALLINGFORD, CONN.<br>ENGINEER | | CE # 27 660 KA<br>DATE May '79 PAGE C-2 |

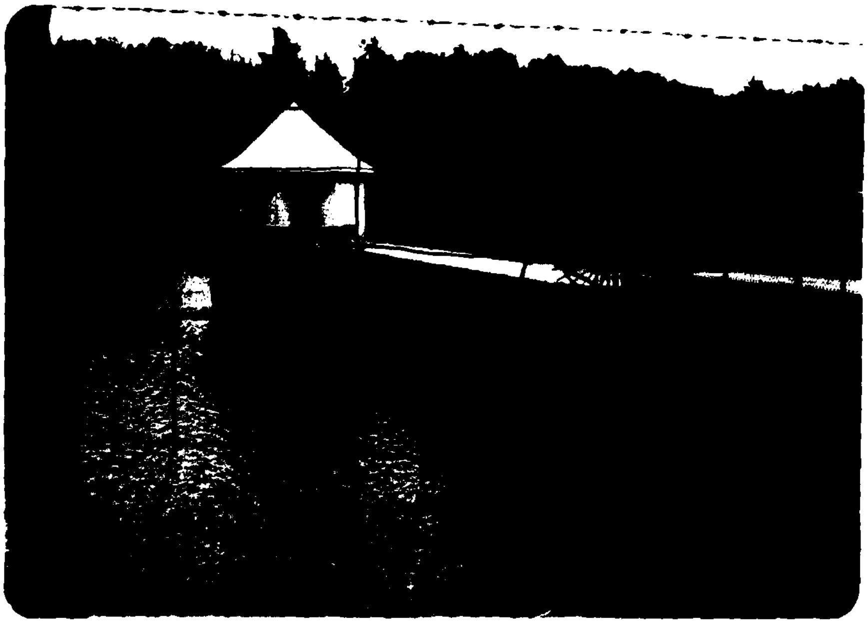


PHOTO 5 - Upstream face of dam and gatehouse from right abutment.

PHOTO 6 - Downstream face near left end of dam. Note trees growing in close proximity to exposed toe of dam.

| | | |
|---|---|---|
| US ARMY ENGINEER DIV. NEW ENGLAND<br>CORPS OF ENGINEERS<br>WALTHAM, MASS | NATIONAL PROGRAM OF<br>INSPECTION OF<br>NON-FED. DAMS | Glen Lake Dam<br>Sargent River<br>Woodbridge, Connecticut |
| CAHN ENGINEERS INC.<br>WALLINGFORD, CONN.<br>ENGINEER | | CE# 27 660 KA<br>DATE May '79 PAGE C-3 |

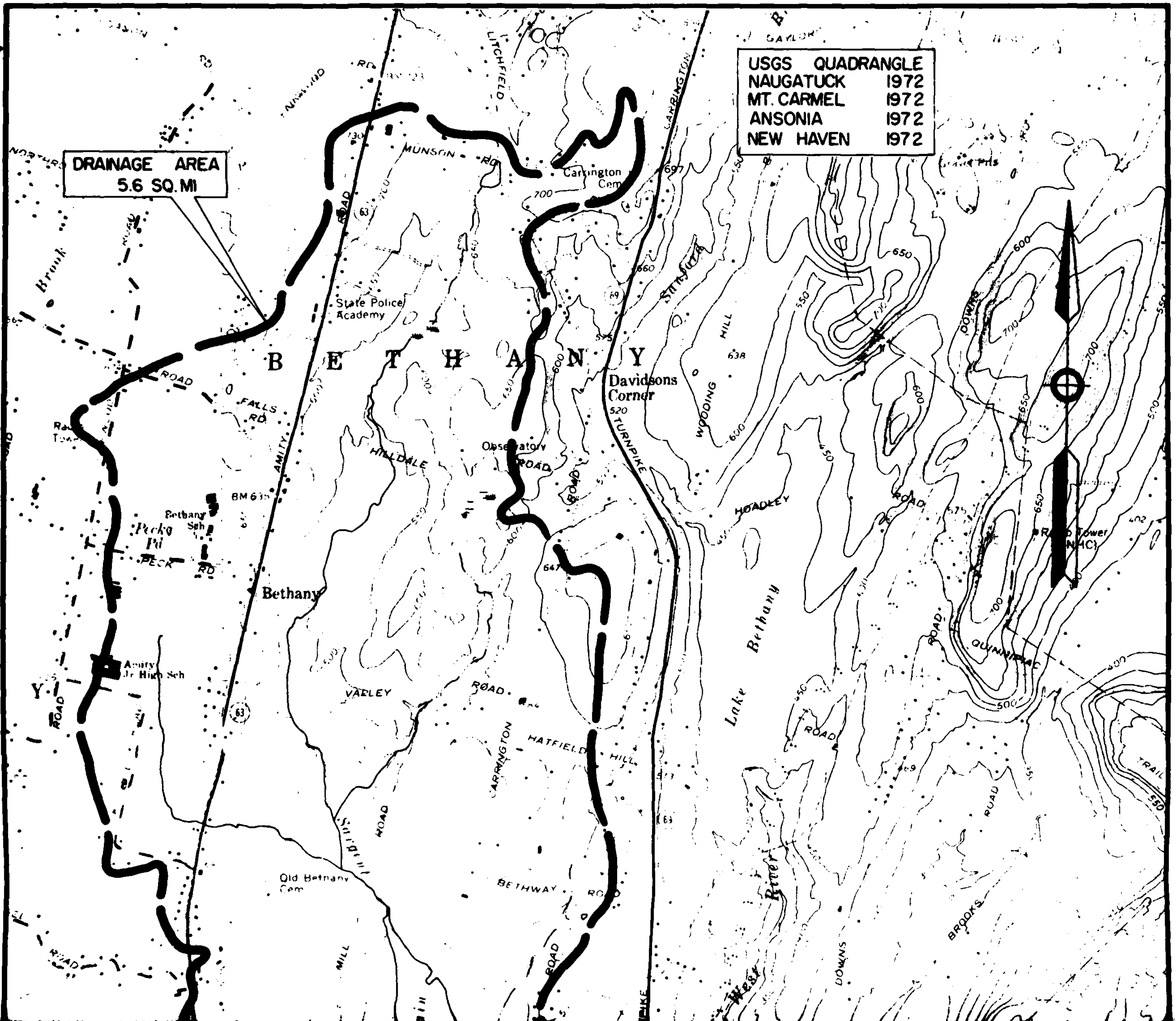
PHOTO 8 - Other [illegible] deterioration [illegible] efflorescence [illegible] downstream fa[illegible]

PHOTO 7 - Deterioration of [illegible] facing, minor [illegible] efflorescence [illegible] on down[illegible]

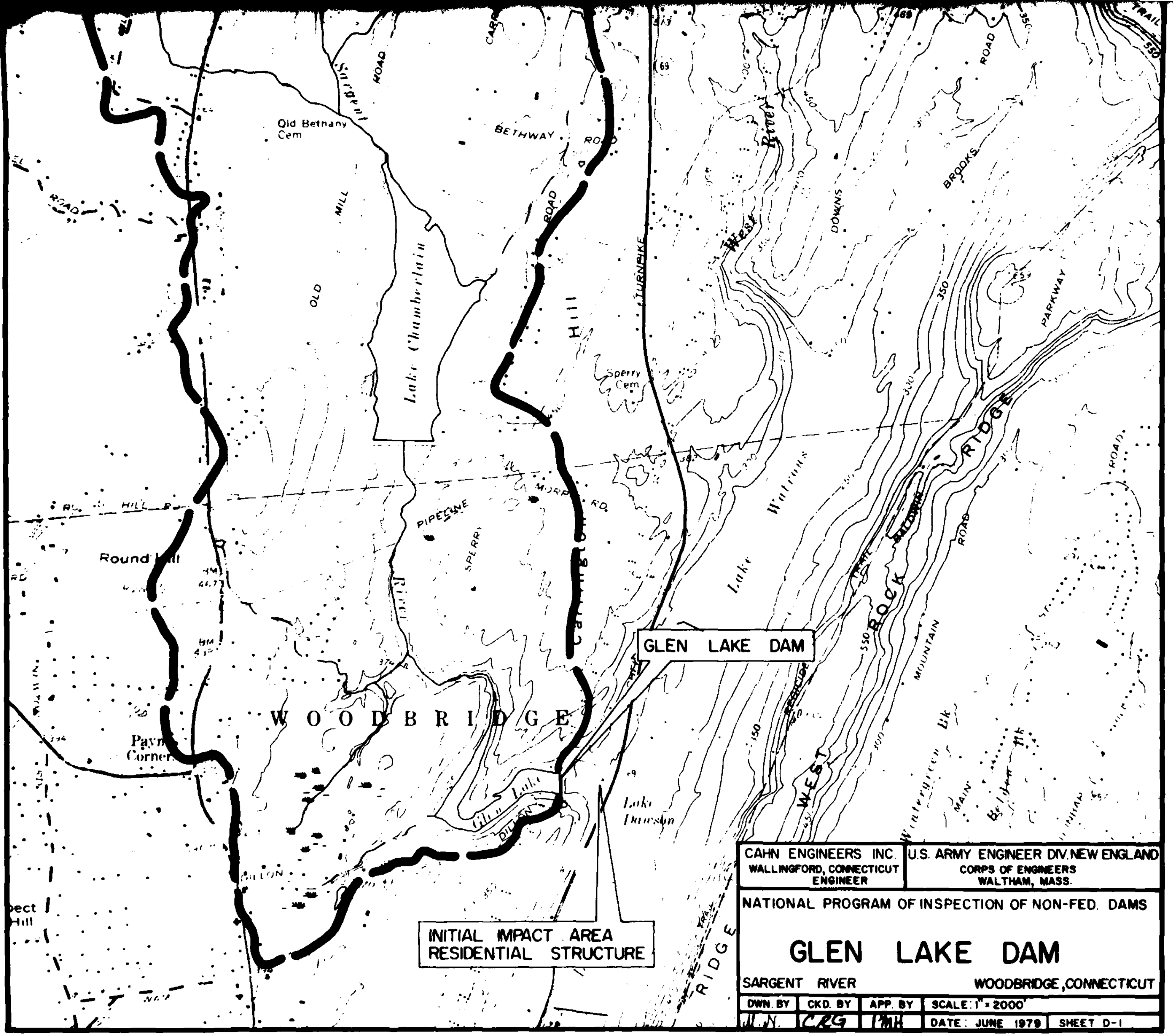
| US ARMY ENGINEER DIV. NEW ENGLAND<br>CORPS OF ENGINEERS<br>WALTHAM, MASS | NATIONAL PROGRAM OF | Glen Lake Dam<br>Sargent River |
|---|---|---|
| CAHN ENGINEERS INC.<br>WALLINGFORD, CONN.<br>ENGINEER | INSPECTION OF<br>NON-FED. DAMS | Woodbridge, Conne[illegible]<br>CE # 27 660 KA<br>DATE May '79 PAGE [illegible] |

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS



USGS QUADRANGLE
NAUGATUCK 1972
MT. CARMEL 1972
ANSONIA 1972
NEW HAVEN 1972
DRAINAGE AREA
5.6 SQ. MI
BETHANY
Davidsons Corner
State Police Academy
Carrington Cem
Observatory
Bethany
Old Bethany Cem
Lake Bethany
West River
TURNPIKE
AMITY ROAD
MUNSON RD
LITCHFIELD
CARRINGTON
WOODING HILL
HOADLEY ROAD
DOWNS ROAD
QUINNIPIAC
BROOKS ROAD
HATFIELD HILL
BETHWAY ROAD
VALLEY ROAD
HILLDALE
FALLS RD
PECK RD
Amity Jr High Sch
Bethany Sch




| CAHN ENGINEERS INC.<br>WALLINGFORD, CONNECTICUT<br>ENGINEER | U.S. ARMY ENGINEER DIV. NEW ENGLAND<br>CORPS OF ENGINEERS<br>WALTHAM, MASS. | | |
|---|---|---|---|

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

# GLEN LAKE DAM

SARGENT RIVER — WOODBRIDGE, CONNECTICUT

| DWN. BY | CKD. BY | APP. BY | SCALE: 1" = 2000' |
|---|---|---|---|
| J.N. | CRG | PMH | DATE: JUNE 1979 — SHEET D-1 |

Cahn Engineers Inc. Consulting Engineers

Project INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND  Sheet D-1 of 13
Computed By [illegible]  Checked By CRG  Date 5/15/79
Field Book Ref.  Other Refs. CE# 27-660-KA  Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

GLEN LAKE DAM, WOODBRIDGE, CT.

I) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD:

a) WATERSHED CLASSIFIED AS "ROLLING"

b) WATERSHED AREA

i) TOTAL *D.A. = 5.7 SQ MI

ii) D.A. U/S FROM LAKE CHAMBERLAIN: *D.A. = 4.0 SQ MI

iii) DIRECT D.A. TO GLEN LAKE (D/S FROM LAKE CHAMBERLAIN): *D.A. = 1.7 SQ MI

*NOTE: DATA FROM NEW HAVEN WATER CO. (REPORT & DATA BY J.N. CANE) ON THE DAMS ON WEST & SARGENT RIVERS, DATED JUNE, 1905 AND C.E. LAKE CHAMBERLAIN DAM, CT 00306, PHASE I INSPECTION REPORT, AUG 1978.

c) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAX PROBABLE DISCHARGES" - GUIDE CURVE FOR PMF - PEAK FLOW RATES:

i) PMF ≈ 1800 CFS/SQ MI FOR TOTAL DA.

ii) PMF ≈ 1900 CFS/SQ MI FOR LAKE CHAMBERLAIN D.A.

iii) PMF ≈ 2800 CFS/SQ MI FOR DIRECT DA. TO GLEN LAKE (BY EXTRAPOLATION)

# Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Sheet D-2 of 13
Computed By HAL
Checked By CPG
Date 5/15/79
Field Book Ref.
Other Refs. CE#27-660-KA
Revisions

GLEN LAKE DAM

1-Cont'd) MAXIMUM PROBABLE FLOOD

d) PEAK INFLOW

BECAUSE A LARGE PORTION OF THE GLEN LAKE WATERSHED IS REGULATED BY LAKE CHAMBERLAIN (A RESERVOIR OF RELATIVELY LARGE SURFACE AREA AND STORAGE CAPACITY) THE EFFECT OF THIS RESERVOIR ON THE PEAK INFLOW OF GLEN LAKE SHOULD BE CONSIDERED.

PEAK OUTFLOW AT PMF FOR LAKE CHAMBERLAIN IS ESTIMATED AT $Q_{P_3}^3 = 5500$ CFS (C.E. PHASE I INSPECTION REPORT, AUG. 1978).
SIMILARLY, AT ½ PMF, $Q_{P_3}' = 2600$ CFS FOR LAKE CHAMBERLAIN.

THEREFORE, THE PEAK INFLOW TO GLEN LAKE IS ESTIMATED AS FOLLOWS:

i) PEAK OUTFLOW FROM LAKE CHAMBERLAIN @ PMF : $(PMF)_1 = Q_{P_3} = 5500$ CFS

ii) CONTRIBUTION FROM DIRECT D.A. TO GLEN LAKE:
$(PMF)_2 = 1.7 \times 1800 \doteq 3100$ CFS

∴ iii) PEAK INFLOW TO GLEN LAKE:

$$PMF = 5500 + 3100 \doteq \underline{\underline{8600}} \text{ CFS}$$

SIMILARLY, "½ PMF" $= 2600 + 1600 \doteq 4200$ CFS (USE ½ PMF OR $\underline{4300}$ CFS FOR ONES)

THEREFORE, THE STORAGE EFFECT OF LAKE CHAMBERLAIN REDUCES THE PMF PEAK INFLOW OF GLEN LAKE BY (≈) 1700 CFS, AND THE ½ PMF PEAK INFLOW BY (≈) 900 CFS.

# Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION
Computed By [illegible] Checked By [illegible]
Field Book Ref. Other Refs. CE#27-662-KA

Sheet D-3 of 13
Date 5/15/79
Revisions

GLEN LAKE DAM

2) SPILLWAY DESIGN FLOOD (SDF)

a) CLASSIFICATION OF DAM ACCORDING TO NED-ACE RECOMMENDED GUIDELINES:

i) SIZE: STORAGE (MHW) $\doteq$ 710 AC-FT (50 < S < 1000 AC-FT)
HEIGHT $\doteq$ 62' (40 < H < 100 FT)

STORAGE: FROM NEW HAVEN WATER CO. SUMMARY DATA SHEET BY J.H.C. DATED JUNE, 1965 AND INVENTORY (STATISTICS ON DAMS) DATED 8/12/74, RESERVOIR TOTAL STORAGE AT FLOWLINE (SPILLWAY CREST ELEV. (*) 218.3' MSL* = 215' MHW) S = 482 AC-FT (157 MG)
AREA @ FLOWLINE: A = 23 AC; C.E. MEAS. AREA @ CONTOUR 230' MSL, $A_{230} \doteq 28$ AC ∴ USE $A_{AVE} \doteq 25$ AC ∴ $S_{MAX} \doteq 710$ AC-FT TO TOP OF DAM (ELEV. 227.3' MSL = 224' MHW).

HEIGHT: FROM SAME DATA SOURCES AS FOR STORAGE. NATURAL STREAM BED ELEV. 165.3' MSL = 162' MHW. (CREST TO DEEPEST FOUNDATION ±75').

ii) HAZARD POTENTIAL: GLEN LAKE DAM IS LOCATED U/S FROM LAKE DAWSON DAM, KONOLDS POND AND OF URBAN DEVELOPMENTS IN WOODBRIDGE, CT. IN PARTICULAR, ONE LOW HOUSE IS IMMEDIATELY D/S FROM GLEN LAKE DAM AND AT LEAST TWO STRUCTURES ARE IMMEDIATELY D/S FROM LAKE DAWSON DAM, INCLUDING A WATER TREATMENT PLANT.

*NOTE: ELEVATIONS GIVEN IN NEW HAVEN WATER CO. DATA ARE NEW HAVEN DATUM (MHW).
USC&GS DATUM (MSL) = NEW HAVEN DATUM (MHW) + 3.31'
(USE +3.3')

**Cahn Engineers Inc.** Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION | Sheet D-4 of 13
Computed By HWL | Checked By CRS | Date 5/16/79
Field Book Ref. | Other Refs CE#37-663-KA | Revisions

GLEN LAKE DAM

2.a-(Cont'd) CLASSIFICATION OF DAM ACCORDING TO NED-ACE GUIDELINES

iii) CLASSIFICATION:

SIZE : INTERMEDIATE
HAZARD : HIGH

b) SDF = PMF = 8600 $^{cfs}$ $\qquad$ ½ PMF = 4300 $^{cfs}$

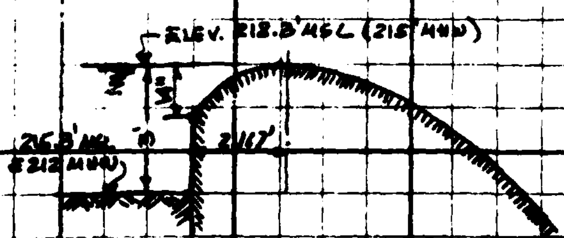
3) SURCHARGE AT PEAK INFLOW

a) PEAK INFLOW : $Q_p = 8600^{cfs}$ $\qquad$ $Q_p' = \frac{1}{2} PMF = 4300^{cfs}$

b) SPILLWAY (OUTFLOW) RATING CURVE

i) SPILLWAY:

THE GLEN LAKE DAM SPILLWAY IS AN OGEE (WES) TYPE SPILLWAY, WITH 40' LONG CREST AT (*) ELEV. 218.3' MSL (215' MWW). THE HEIGHT BETWEEN THE SPILLWAY CREST AND THE TOP OF THE DAM IS H=9'. A CONCRETE SLAB DECK, FOOT BRIDGE WITH CHAIN LINK FENCE RAILINGS CROSSES THE SPILLWAY (LOW CHORD ELEV. ≐ TOP OF DAM ELEV. = 227.3' MSL ≐ 224' MWW). THE DEPTH OF THE APPROACH CHANNEL TO THE CREST OF THE SPILLWAY IS P=3'.




(*) DATA FROM THE NEW HAVEN WATER CO. INVENTORY (STATISTICS OF DAMS) SHEETS DATED 8/12/74 AND RECORD DRAWINGS BY MALCOLM PIRNIE ENGS. OF THE CONTOUR OF THE SPILLWAY, DATED DEC. 1967 (REVISED 6/5/73)

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Project NON-FEDERAL DAMS INSPECTION Sheet D-5 of 13
Computed By HLL Checked By CRG Date 5/16/79
Field Book Ref. Other Refs. CE #27-660-KA Revisions

GLEN LAKE DAM

3.b - (CONT'D) OUTFLOW RATING CURVE

∴ ASSUME SPILLWAY DISCHARGE COEFFICIENT: $C \doteq 3.7$ FOR THE EXPECTED RANGE OF SURCHARGE, ($P/H_o \doteq 0.3$).

USING THE CREST ELEVATION AS DATUM (ELEV. 218.3' MSL = 215' MMW), THE SPILLWAY DISCHARGE IS APPROXIMATED BY:

$$Q_c \doteq 150\,H^{3/2}$$

ii) EXTENSION OF RATING CURVE FOR SURCHARGE HEADS ABOVE TOP OF DAM.

THE DAM IS A CONCRETE MASONRY GRAVITY DAM (±) 340' LONG (EXCLUDING THE SPILLWAY). THE TOP OF THE DAM AT ELEV. 227.3' MSL (224' MMW) IS (±) 8.2' WIDE. TO THE LEFT, A LOW AND WIDE DIKE (±5'H × 80'W × ±160' LONG) CLOSES A DEPRESSION AGAINST THE SLOPING TERRAIN WHICH RISES (±) 15' IN A DISTANCE OF (±) 40'. TO THE RIGHT, THE TERRAIN CONTINUES ESSENTIALLY FLAT FOR (±) 20' AND THEN RISES AT (±) 3" TO 1" SLOPE. THE SLOPING TERRAIN IS WOODED MOSTLY WITH EVERGREENS.

ASSUME $C = 2.7$ FOR FLOW OVER THE DAM AND FLAT TERRAIN
$C = 2.5$ FOR THE SLOPING TERRAIN

THE EXPECTED REDUCTION IN SPILLWAY CAPACITY AT HEADS ABOVE THE TOP OF THE DAM ($H \geq 9'$) RESULTING FROM LOWER P/H RATIOS AND BRIDGE OBSTRUCTIONS, ARE CONSIDERED RELATIVELY SMALL AND THEREFORE, NEGLECTED.

ASSUME ALSO, EQUIVALENT LENGTHS FOR THE SLOPING TERRAIN AT THE SIDES OF THE DAM AS FOLLOWS:

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Project NON-FEDERAL DAMS INSPECTION — Sheet D-6 of 13
Computed By HLL — Checked By CPG — Date 5/16/79
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GLEN LAKE DAM

3.6 - (Cont'd) OUTFLOW RATING CURVE

$$L_2' \cong \frac{2}{3}\left(\frac{3}{1}\right)(H-9) = 2(H-9) \quad \therefore \quad Q_2' = 5(H-9)^{5/2}$$

$$L_1' \cong \frac{2}{3}\left(\frac{40}{5}\right)(H-9) = 5.3(H-9) \quad \therefore \quad Q_1' = 13.3(H-9)^{5/2}$$

WHERE $Q_2'$ & $Q_1'$ ARE THE RESULTING FLOW EXPRESSIONS FOR THE SLOPING TERRAIN.

THEREFORE, THE TOTAL OVERFLOW RATING CURVE MAY BE APPROXIMATED BY:

$$Q = 150H^{3/2} + 1400(H-9)^{3/2} + 18(H-9)^{5/2}$$

THE OUTFLOW RATING CURVE IS PLOTTED ON NEXT PAGE.

c) SPILLWAY CAPACITY TO TOP OF DAM:

$H = 9'$ $\therefore$ $Q_S \cong 4100^{cfs}$ (($\pm$)48% of $Q_{P_1}$; ($\pm$)95% of $Q_{P_1}'$)

d) SURCHARGE HEIGHT TO PASS ($Q_{P_1}$):

i) @ $Q_{P_1}$ = PMF = $8600^{cfs}$ $H_1 \cong 10.7'$

ii) @ $Q_{P_1}'$ = ½ PMF = $4300^{cfs}$ $H_1' \cong 9.2'$

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Project NON-FEDERAL DAMS INSPECTION — Sheet D-7 of 13
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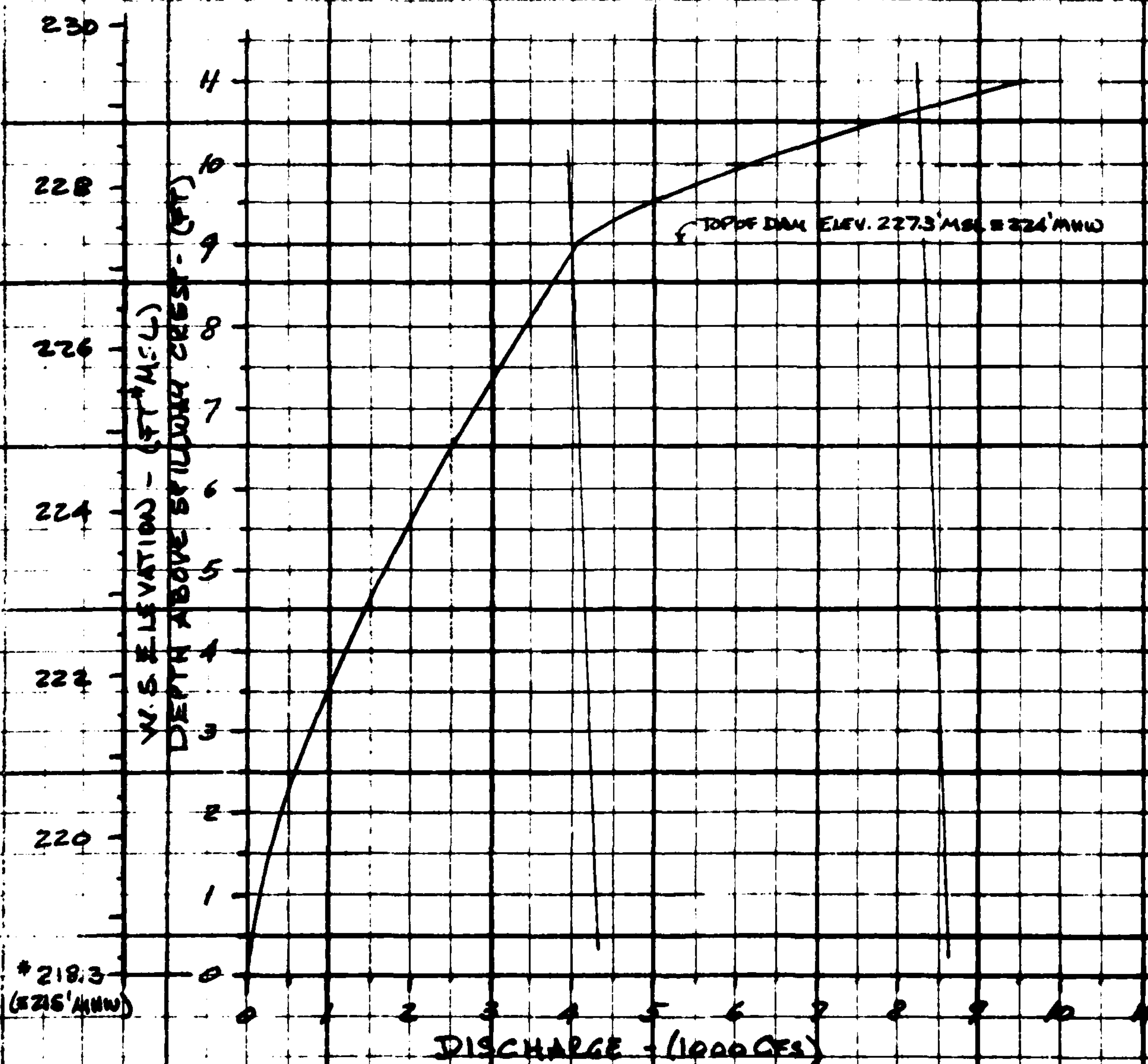
GLEN LAKE DAM

3-(Cont'd) OUTFLOW RATING CURVE




*SPILLWAY CREST ELEV. 218.3' MSL ≡ 215' MHW. -

MSL (USGS DATUM) ≡ MHW (NEW HAVEN WATER CO. DATUM) + 3.3'

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Project NON-FEDERAL DAMS INSPECTION Sheet D-8 of 13
Computed By HLL Checked By CPG Date 5/16/79
Field Book Ref. Other Refs. CE#27-660-KA Revisions

GLEN LAKE DAM

4) EFFECT OF SURCHARGE STORAGE ON MAX. PROBABLE DISCHARGE (OUTFLOW)

a) RESERVOIR (LAKE) AREA @ FLOW LINE. $^*A_0 \approx 23^{AC}$

∴ ASSUME AVG. LAKE AREA WITHIN EXPECTED SURCHARGE: $^*A_{avg} \approx 25^{AC}$

* SEE "STORAGE" p.3 OF THESE COMPUTATIONS.

b) ASSUME NORMAL POOL LEVEL (±) 0.5' ABOVE SPILLWAY CREST (EL. 218.8' MSL)

c) WATERSHED AREA: D.A. = 5.7 $^{sq\,mi}$ (See p.1)

d) DISCHARGE ($Q_{p_2}$) AT VARIOUS HYPOTHETICAL SURCHARGE ELEVATIONS:

$H = 10'$ $V = 25(10-0.5) \approx 238^{AC\,FT}$ ∴ $S = \frac{238}{5.7 \times 53.3} = 0.78''$

$H = 5'$ $V \approx 113^{AC\,FT}$ ∴ $S = 0.37''$

∴ FROM APPROXIMATE STORAGE ROUTING NED-ACE GUIDELINES (19" MAX. PROBABLE R.O. IN NEW ENGLAND):

$Q_{p_2} = Q_{p_1}\left(1 - \frac{S}{19}\right)$ AND FOR ½ PMF: $Q'_{p_2} = Q'_{p_1}\left(1 - \frac{S}{9.5}\right)$

∴ FOR THE ABOVE HYPOTHETICAL SURCHARGES:

$H = 10'$ $Q_{p_2} \approx 8250^{CFS}$ $Q'_{p_2} = 3950^{CFS}$

$H = 5'$ $Q_{p_2} = 8430^{CFS}$ $Q'_{p_2} = 4130^{CFS}$

ACTUALLY, FOR $H = 0.5$; $Q_{p_2} = 8600^{CFS}$, $Q'_{p_2} = 4300^{CFS}$

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Project NON-FEDERAL DAMS INSPECTION
Sheet D-9 of 13
Computed By HM
Checked By CRG
Date 5/16/79
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GLEN LAKE DAM

4-(Cont'd) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOW:

e) PEAK OUTFLOW ($Q_{P_3}$)

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD (SEE p. 7 OF THESE COMPS)

$Q_{P_3} \approx 8220^{CFS}$ $\quad H_3 \approx 10.6'$ $\quad$ FOR $Q_{P_1} = PMF$

$Q'_{P_3} \approx 3990^{CFS}$ $\quad H'_3 \approx 8.9'$ $\quad$ FOR $Q'_{P_1} = \frac{1}{2}PMF$

f) SPILLWAY CAPACITY RATIO TO OUTFLOW

SPILLWAY CAPACITY TO TOP OF DAM: $Q_S \approx 4100^{CFS}$

∴ SPILLWAY CAPACITY IS (≈)50% THE OUTFLOW @ PMF AND (≈)103% THE OUTFLOW @ ½ PMF

5) SUMMARY:

a) PEAK INFLOW: $Q_{P_1} = PMF = 8600^{CFS}$ $\quad Q'_{P_1} = \frac{1}{2}PMF = 4300^{CFS}$

b) PEAK OUTFLOW: $Q_{P_3} \approx 8220^{CFS}$ $\quad Q'_{P_3} \approx 3990^{CFS}$

c) SPILLWAY MAX. CAPACITY: $Q_S \approx 4100^{CFS}$ OR (≈)50% OF $Q_{P_3}$ AND (≈)103% OF $Q'_{P_3}$

THEREFORE, AT SDF = PMF THE DAM IS OVERTOPPED (≈)1.6' (W.S. EL. 228.9' NGVD = EL. 225.[illegible]) OR, TO A SURCHARGE OF (≈)10.6' ABOVE THE SPILLWAY. AT A TEST FLOOD $Q'_P = \frac{1}{2}PMF$, THE SPILLWAY MAY PASS THE TEST FLOOD WITH PRACTICALLY NO FREEBOARD TO THE TOP OF THE DAM OR, TO A SURCHARGE OF (≈)8.9' ABOVE THE SPILLWAY CREST.

GLEN LAKE DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE IMMEDIATELY D/S FROM DAM:

a) BREACH WIDTH:

i) MID-HEIGHT (±) ELEV. 196.3' MSL (193' MMW) $(227.3 - \frac{62}{2} = 196.3' MSL)$
"SEE "HEIGHT" p.3 OF THESE COMPS.

∴ ii) APPROX. MID-HEIGHT LENGTH: $L_m = 190'$ ((±) FROM C.E. SURVEY MAP)

∴ iii) BREACH WIDTH (SEE NED ACE D/S DAM FAILURE GUIDELINES):

$W = 0.4 \times 190 = 76'$ ∴ ASSUME $W_b = \underline{70'}$

b) PEAK FAILURE OUTFLOW $(Q_p)$

ASSUME SURCHARGE TO TOP OF DAM; THEREFORE.

i) HEIGHT AT TIME OF FAILURE: $Y_0 = 62'$

ii) SPILLWAY DISCHARGE: $Q_s = 4100$ CFS (SEE p.6 OF THESE COMPS.)

iii) BREACH OUTFLOW $(Q_b)$:

$Q_b = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2} = 57500$ CFS

iv) PEAK FAILURE OUTFLOW $(Q_p)$: $Q_p = Q_s + Q_b = \underline{61600}$ CFS

c) FLOOD WAVE HEIGHT IMMEDIATELY D/S FROM DAM:

$y = 0.44\, Y_0 = \underline{27'}$

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Project NON-FEDERAL DAMS INSPECTION — Sheet D-11 of 13
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GLEN LAKE DAM

2) ESTIMATE OF D/S DAM FAILURE CONDITIONS AT IMPACT AREA:

(SEE NED-ACE GUIDELINES FOR ESTIMATING D/S DAM FAILURE HYDROGRAPHS)

GLEN LAKE DAM IS LOCATED (±)900' U/S FROM LAKE DAWSON. THE CHANNEL BETWEEN THE TWO RESERVOIRS IS CONSIDERED TOO SHORT SO AS TO CAUSE ANY APPRECIABLE MODIFICATION TO A FLOOD WAVE RESULTING FROM THE FAILURE OF GLEN LAKE DAM.

THEREFORE, FOR THIS ANALYSIS, THE PEAK INFLOW TO LAKE DAWSON FROM THE FLOOD WAVE PRODUCED AT GLEN LAKE DAM WILL BE ASSUMED TO BE:

$$Q_{p} = \underline{61600}^{\text{CFS}}$$

a) W.L. RAISE AND OUTFLOW OF LAKE DAWSON BECAUSE OF FAILURE OF GLEN LAKE DAM:

i) VOLUME OF STORAGE AT TIME OF FAILURE (GLEN) $S \approx 710^{\text{AC-FT}}$
(SEE p.3 OF THESE COMPS. - RES. FULL TO TOP OF DAM)

ii) LAKE DAWSON DATA (FROM NEW HAVEN WATER CO. FILES & C.E. FIELD OBSERVATIONS & SURVEY)

LENGTH OF SPILLWAY: $L = 110'$ ASSUME $C = 3.6'$ (ELEV. 158.3' MSL)
HEIGHT TO TOP OF DAM: $H = 7'$ (ELEV. 158.3' MSL TO ELEV. 165.3' MSL)
LENGTH OF DAM EXCL. SPILLWAY (±)800' (ASSUME C=3.0 IF OVERTOPPED)
TERRAIN AT LEFT RISES 30' IN (±)150' AND AT RIGHT RISES 10' IN (±)800'
(ASSUME C=2.8 IF OVERTOPPED). FLASHBOARDS (2.5' HIGH) USAGE DISCONTINUED, [illegible]
AVG. LAKE AREA WITHIN EXPECTED SURCHARGE: $A_{AVG} \approx 74^{\text{AC}}$

∴ EQUIV. LENGTH OF SLOPING SIDES - ASSUMED TO BE APPROX.:

$$L'_{EQL} = \frac{2}{3}\left(\frac{800}{10} + \frac{150}{30}\right)(H-7) = 56.7\,(H-7) \qquad (\text{FOR } H \leq 17')$$

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Project NON-FEDERAL DAMS INSPECTION | Sheet D-12 of 13
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Field Book Ref. | Other Refs. CE #27-660-KA | Revisions

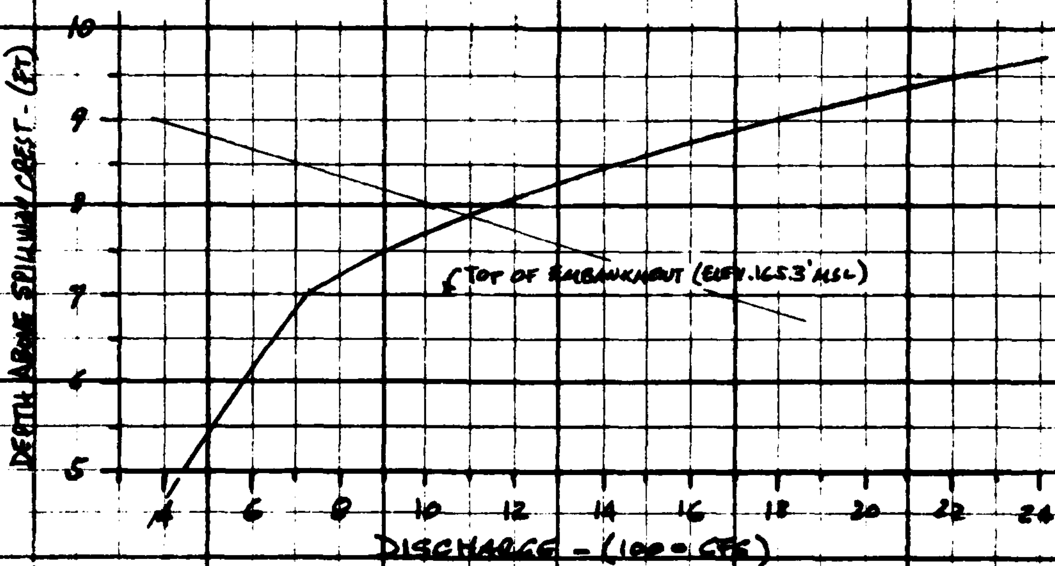
GLEN LAKE DAM

2.a - Cont'd) D/S DAM FAILURE CONDITIONS - (LAKE DAWSON DATA)

∴ THE LAKE DAWSON OUTFLOW CAN BE APPROXIMATED BY:

$$Q_{OUT.} \doteq 400H^{3/2} + 2400(H-7)^{3/2} + 160(H-7)^{5/2} \quad (\text{FOR } H \leq 17')$$

iii) LAKE DAWSON OVERFLOW RATING CURVE W/O FLASHBOARDS:




b) LAKE DAWSON OUTFLOW:

NOTE: BECAUSE THE LAKE DAWSON'S SURCHARGE STORAGE FOR $H_1 = 12.6'$ @ $Q_{P_1} = 6,600^{CFS}$ IS TOO LARGE ($V \doteq 930^{AC\text{-}FT}$) IN COMPARISON TO THE MAX. VOLUME OF STORAGE RELEASED UPON FAILURE OF GLEN LAKE ($S \doteq 70^{AC\text{-}FT}$), THE FLOOD ROUTING PROCEDURE ORIGINALLY GIVEN BY THE NED-ACE GUIDELINES IS NOT SUITABLE FOR THIS APPLICATION. THEREFORE, THE LAKE DAWSON OUTFLOW WILL BE ESTIMATED FOLLOWING A SIMILAR APPROACH

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Project NON-FEDERAL DAMS INSPECTION — Sheet D-13 of 13
Computed By HW — Checked By CRG — Date 5/21/79
Field Book Ref. — Other Refs. CE#27-660-KA — Revisions

GLEN LAKE DAM

2.6 - Cont'd) D/S DAM FAILURE CONDITIONS - (LAKE DAWSON OUTFLOW)

DUE TO THAT USED IN ROUTING THE PMF TEST FLOOD THROUGH GLEN LAKE (SEE PP. 7-9 OF THESE COMPUTATIONS)

i) DISCHARGE ($Q_{p2}$) AT VARIOUS HYPOTHETICAL SURCHARGES (DAWSON):

$H = 9'$ $V_1 = 74 \times 9 = 666^{AC\,FT}$ $\therefore Q_{p2} = 61600\left(1 - \frac{666}{[illegible]}\right) \approx 3800^{CFS}$

$H = 8'$ $V_1 \approx 592^{AC\,FT}$ $\therefore Q_{p2} \approx 10200^{CFS}$

$H = 7'$ $V = 518^{AC\,FT}$ $\therefore Q_{p2} \approx 16700^{CFS}$

ii) LAKE DAWSON OUTFLOW ($Q_{p3}$) AND SURCHARGE ($H_3$):

$\therefore Q_{p3} \approx 10800^{CFS}$ $H_3 \approx 7.9'$ (SEE P. 12 OF THESE COMPS)

$\therefore$ UPON FAILURE OF GLEN LAKE DAM, LAKE DAWSON DAM WOULD BE OVERTOPPED BY (±) 0.9' WITH A RESULTING OUTFLOW OF (±) $Q_{p3} = 10800^{CFS}$ TO THE IMPACT AREA (HOUSE) JUST D/S FROM LAKE DAWSON.

3) SUMMARY

a) PEAK FAILURE OUTFLOW: $Q_p = 61600^{CFS}$ STAGE: $Y_1 \approx 27'$
(HOUSE (±) 700' D/S FROM GLEN LAKE W/ FIRST FLOOR* (±) 15' ABOVE THE STREAM BED)

b) LAKE DAWSON DAM OUTFLOW $Q_{p3} = 10800^{CFS}$
SURCHARGE AT LAKE DAWSON: $H_3 \approx 7.9'$ ABOVE SPWY (± 0.9' OVERTOPPING THE EARTH EMBANKMENT)
(BUILDING* (±) 200' D/S FROM LAKE DAWSON WITH F.F. ELEV. (±) 135' MSL - i.e., (±) 30' LOWER THAN LAKE DAWSON DAM TOP OF EMBANKMENT (±) ELEV. 165.5 MSL).

* DATA FROM U.S.G.S. NEW HAVEN QUADRANGLE SHEET SCALE 1:24000, 1967 PHOTOREV. 1972.

# PRELIMINARY GUIDANCE

# FOR ESTIMATING

# MAXIMUM PROBABLE DISCHARGES

# IN

# PHASE I DAM SAFETY

# INVESTIGATIONS

New England Division
Corps of Engineers

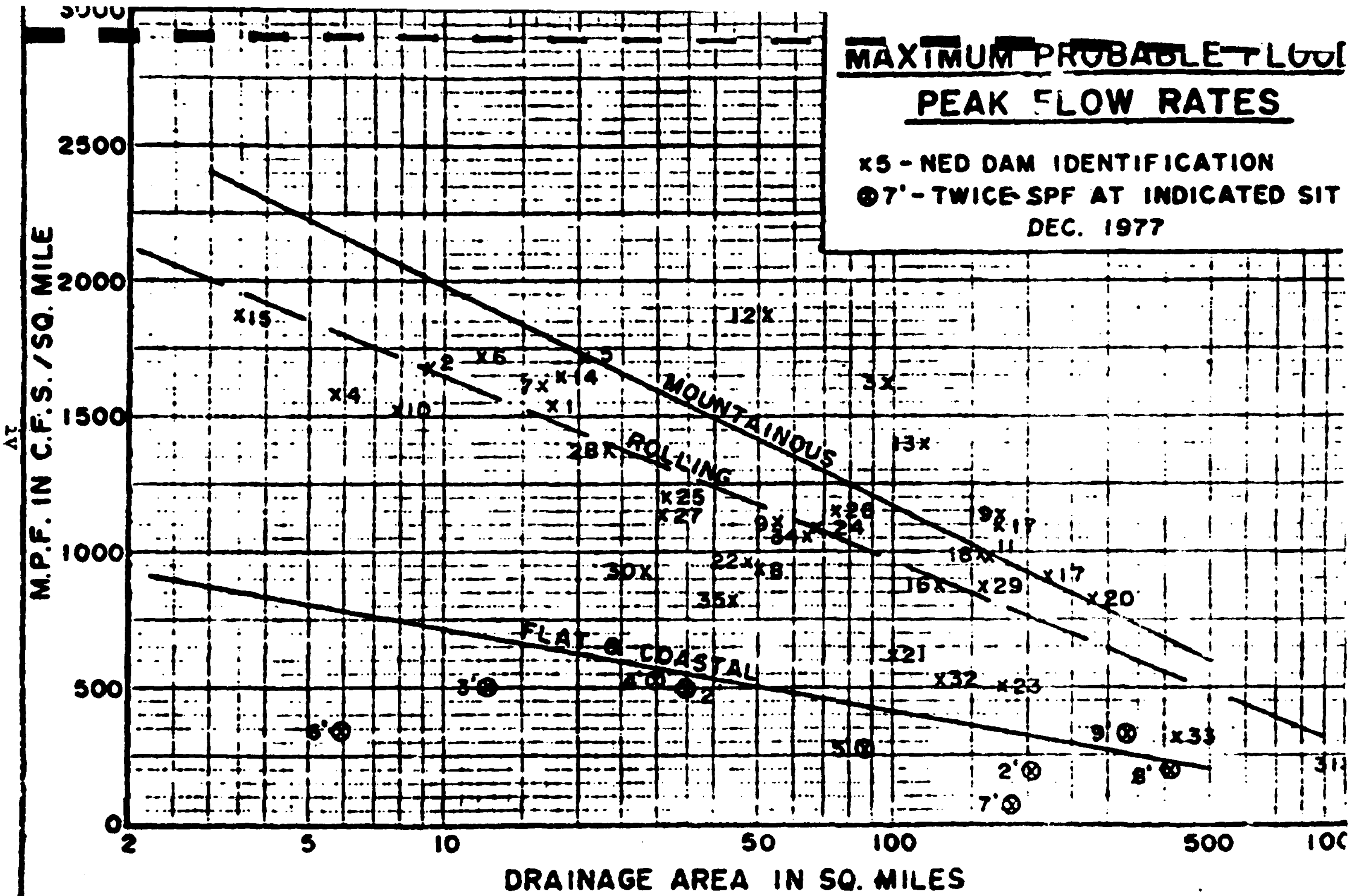
March 1978

## MAXIMUM PROBABLE FLOOD INFLOWS
## NED RESERVOIRS

| Project | Q (cfs) | D.A. (sq. mi.) | MPF cfs/sq. mi. |
|---|---|---|---|
| 1. Hall Meadow Brook | 26,600 | 17.2 | 1,546 |
| 2. East Branch | 15,500 | 9.25 | 1,675 |
| 3. Thomaston | 158,000 | 97.2 | 1,625 |
| 4. Northfield Brook | 9,000 | 5.7 | 1,580 |
| 5. Black Rock | 35,000 | 20.4 | 1,715 |
| 6. Hancock Brook | 20,700 | 12.0 | 1,725 |
| 7. Hop Brook | 26,400 | 16.4 | 1,610 |
| 8. Tully | 47,000 | 50.0 | 940 |
| 9. Barre Falls | 61,000 | 55.0 | 1,109 |
| 10. Conant Brook | 11,900 | 7.8 | 1,525 |
| 11. Knightville | 160,000 | 162.0 | 987 |
| 12. Littleville | 98,000 | 52.3 | 1,870 |
| 13. Colebrook River | 165,000 | 118.0 | 1,400 |
| 14. Mad River | 30,000 | 18.2 | 1,650 |
| 15. Sucker Brook | 6,500 | 3.43 | 1,895 |
| 16. Union Village | 110,000 | 126.0 | 873 |
| 17. North Hartland | 199,000 | 220.0 | 904 |
| 18. North Springfield | 157,000 | 158.0 | 994 |
| 19. Ball Mountain | 190,000 | 172.0 | 1,105 |
| 20. Townshend | 228,000 | 106.0(278 total) | 820 |
| 21. Surry Mountain | 63,000 | 100.0 | 630 |
| 22. Otter Brook | 45,000 | 47.0 | 957 |
| 23. Birch Hill | 88,500 | 175.0 | 505 |
| 24. East Brimfield | 73,900 | 67.5 | 1,095 |
| 25. Westville | 38,400 | 99.5(32 net) | 1,200 |
| 26. West Thompson | 85,000 | 173.5(74 net) | 1,150 |
| 27. Hodges Village | 35,600 | 31.1 | 1,145 |
| 28. Buffumville | 36,500 | 26.5 | 1,377 |
| 29. Mansfield Hollow | 125,000 | 159.0 | 786 |
| 30. West Hill | 26,000 | 28.0 | 928 |
| 31. Franklin Falls | 210,000 | 1000.0 | 210 |
| 32. Blackwater | 66,500 | 128.0 | 520 |
| 33. Hopkinton | 135,000 | 426.0 | 316 |
| 34. Everett | 68,000 | 64.0 | 1,062 |
| 35. MacDowell | 36,300 | 44.0 | 825 |

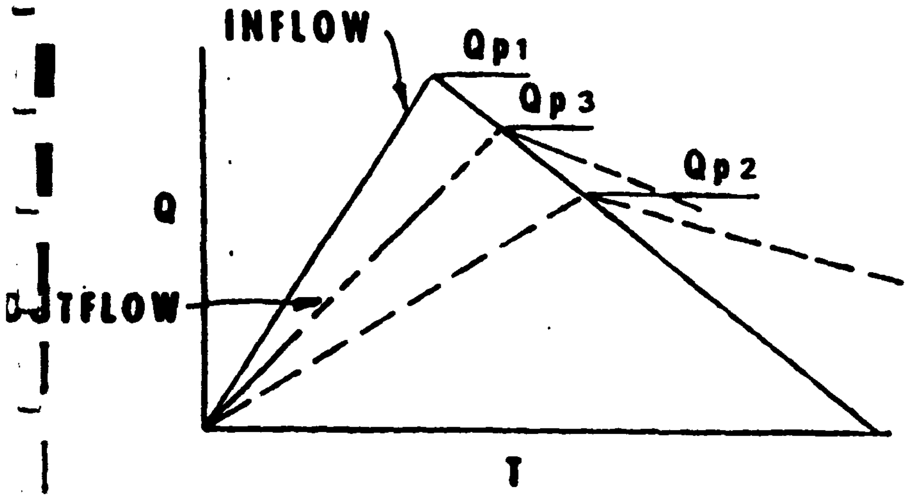
## MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

| | River | SPF (cfs) | D.A. (sq. mi.) | MPF (cfs/sq. mi.) |
|---|---|---|---|---|
| 1. | Pawtuxet River | 19,000 | 200 | 190 |
| 2. | Mill River (R.I.) | 8,500 | 34 | 500 |
| 3. | Peters River (R.I.) | 3,200 | 13 | 490 |
| 4. | Kettle Brook | 8,000 | 30 | 530 |
| 5. | Sudbury River. | 11,700 | 86 | 270 |
| 6. | Indian Brook (Hopk.) | 1,000 | 5.9 | 340 |
| 7. | Charles River. | 6,000 | 184 | 65 |
| 8. | Blackstone River. | 43,000 | 416 | 200 |
| 9. | Quinebaug River | 55,000 | 331 | 330 |



MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SIT
DEC. 1977
MOUNTAINOUS
ROLLING
FLAT & COASTAL
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
2500
2000
1500
1000
500
0
2
5
10
50
100
500

# ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES




STEP 1: Determine Peak Inflow ($Q_{p1}$) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "$Q_{p1}$".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "$Q_{p2}$"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "$Q_{p3}$".

## SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "$Qp_2$"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "$Qp_3$".

c. If Surcharge Height for $Qp_3$ and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "$Qp_3$"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "$Qp_4$"

c. Surcharge Height for $Qp_4$ and "New $STOR_{Avg}$" should Agree closely

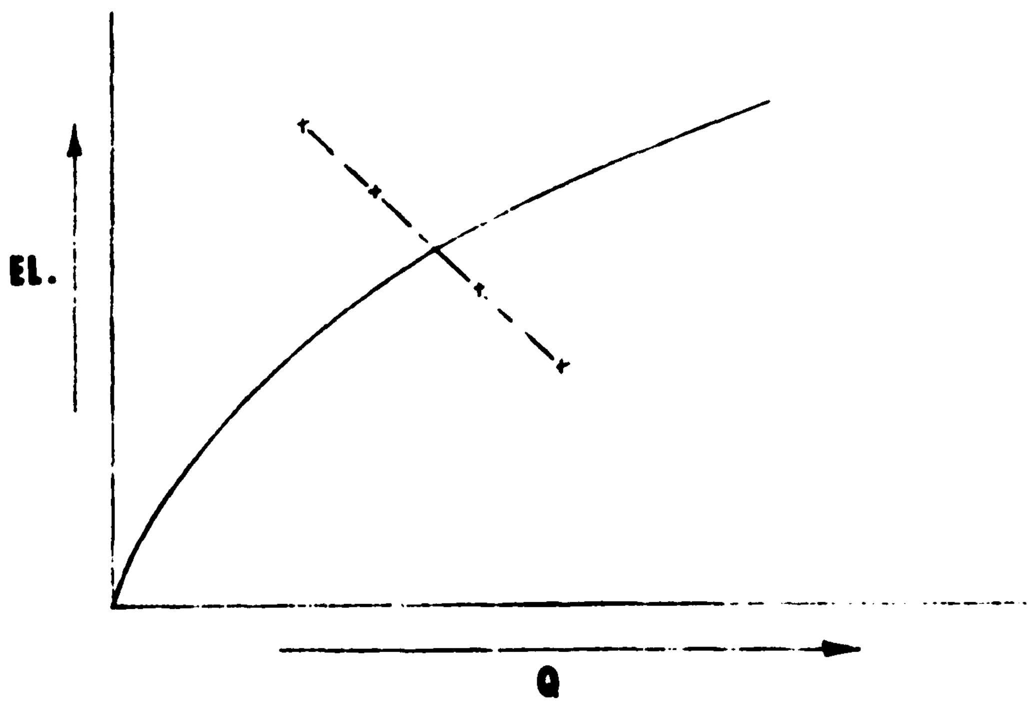
# SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

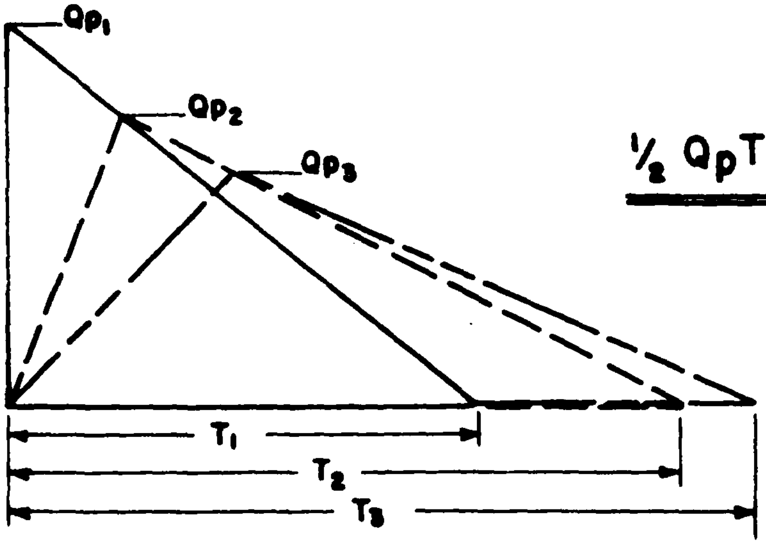
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN $Q_{p1}$ AND 19'' R.O.

| $Q_{p2}$ | STOR | EL. |
|---|---|---|
| ____ | ____ | ____ |
| ____ | ____ | ____ |
| ____ | ____ | ____ |
| ____ | ____ | ____ |
| ____ | ____ | ____ |



# "RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS




$\frac{1}{2} Q_p T = 12\,S$

**STEP 1:** DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

**STEP 2:** DETERMINE PEAK FAILURE OUTFLOW ($Q_{p1}$).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$$

$W_b$ = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

$Y_o$ = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

**STEP 3:** USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

**STEP 4:** ESTIMATE REACH OUTFLOW ($Q_{p2}$) USING FOLLOWING ITERATION.

A. APPLY $Q_{p1}$ TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME ($V_1$) IN REACH IN AC-FT. (NOTE: IF $V_1$ EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL $Q_{p2}$.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE $V_2$ USING $Q_{p2}$ (TRIAL).

D. AVERAGE $V_1$ AND $V_2$ AND COMPUTE $Q_{p2}$.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{ave}}{S}\right)$$

**STEP 5:** FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

# INVENTORY OF DAMS IN THE UNITED STATES

| STATE | IDENTITY NUMBER | DIVISION | STATE | COUNTY | CONGR. DIST. | STATE | COUNTY | CONGR. DIST. | NAME | LATITUDE (NORTH) | LONGITUDE (WEST) | REPORT DATE DAY MO YR |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CT | 317 | NED | CT | 009 | 03 | | | | GLEN LAKE DAM | 4122.6 | 7258.7 | 31AUG79 |

| POPULAR NAME | NAME OF IMPOUNDMENT |
|---|---|
| | GLEN LAKE |

| REGION | BASIN | RIVER OR STREAM | NEAREST DOWNSTREAM CITY-TOWN-VILLAGE | DIST FROM DAM (MI.) | POPULATION |
|---|---|---|---|---|---|
| 01 | 07 | SARGENT RIVER | WOODBRIDGE | 2 | 4000 |

| TYPE OF DAM | YEAR COMPLETED | PURPOSES | STRUCTURAL HEIGHT (FT.) | HYDRAULIC HEIGHT (FT.) | IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.) | IMPOUNDING CAPACITIES NORMAL (ACRE-FT.) | DIST | OWN | FED R | PRV/FED | SCS A | VER/DATE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CTPG | 1907 | S | 75 | 62 | 710 | 482 | NED | N | N | N | N | |

| REMARKS |
|---|
| |

| D/S HAS | SPILLWAY CREST LENGTH | SPILLWAY TYPE | SPILLWAY WIDTH (FT.) | MAXIMUM DISCHARGE (FT.) | VOLUME OF DAM (CY) | POWER CAPACITY INSTALLED (MW) | POWER CAPACITY PROPOSED (MW) | NAVIGATION LOCKS NO | LENGTH (FT.) | WIDTH (FT.) | LENGTH (FT.) | WIDTH (FT.) | LENGTH (FT.) | WIDTH (FT.) | LENGTH (FT.) | WIDTH (FT.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 380 | U | 40 | 4100 | | | | | | | | | | | | |

| OWNER | ENGINEERING BY | CONSTRUCTION BY |
|---|---|---|
| NEW HAVEN WATER COMPANY | ALBERT B HILL | NY CONTINENTAL |

| REGULATORY AGENCY DESIGN | CONSTRUCTION | OPERATION | MAINTENANCE |
|---|---|---|---|
| CT WATER RESOURCES | CT WATER RESOURCES | CT WATER RESOURCES | CT WATER RESOURCES |

| INSPECTION BY | INSPECTION DATE DAY MO YR | AUTHORITY FOR INSPECTION |
|---|---|---|
| CAHN ENGINEERS INC | 01MAY79 | PL 92-367 |

| REMARKS |
|---|
| 48-NY CONTINENTAL JEWELL FILT CO AND UPSON AND GRANNIS |